Pavement Management System Implementation in Vietnam

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Executive Summary

A road infrastructure is one national asset developed with huge effort and investment over the long course to meet all individual demand on mobility and requirement of land transportation over the country including integration to some regional or international road networks. The infrastructure is also requested to be taken over to the next generations, so that road functions need to be carefully preserved and maintained for a long duration of road maintenance due to the fact of deterioration much or less occurred in all facilities.

The warning situation of poor soundness and performance of road network in recent years in Vietnam clearly explains for the high demand for a systematic approach of road management and maintenance to avoid the worst scenario of infrastructure in ruin. Road administrators must not only grasp the current conditions of infrastructure under their jurisdiction but also understand its deterioration or transition of condition states in the future that could be obtained by development of pavement forecasting models and formulation of necessary datasets for proper planning. In broader scope, a Plan - Do - Check - Act (PDCA) cycle must be indispensable.

In asset management, the basic principle of road maintenance is to select “right works”, “right places” and “right timings” of maintenance and repair works in order to ensure the best economy over the long course of road maintenance. With this, road operators are encouraged to shift from *ex post fact* maintenance to strategic planned maintenance and there must be strategic decisions and policies for both managing the existing road infrastructure and new development to expand the road network including reconstruction heavy deteriorated pavements. In order to make proper decision, supporting tools and informative data must be crucial.

This research describes the whole process of pavement management practices in Vietnam for years since the initial stage without data and system as well. Approaching to global asset management systems in early 21st century is one of the important milestone to change the awareness of decision makers on the significance of strategic investment and management of the road infrastructure. Much effort had been made for both data works and system ownership and operation. However, the outcomes were quite limited and far from expectation of optimal allocation of the scare budget between the development of the road network by new construction and maintenance of the existing infrastructure to maximize the value of the stock of road infrastructure for the highest outcomes for the citizens. In more critical, these one-size-fits-all systems are incapable of dealing with demands for customization to fit to the local conditions and practices in both technical conditions and institutional features. With big lessons learned, it has been confirmed that

one-finds-one's-own-size systems seem to be the best solution and development a new pavement management system (PMS) is indispensable for Vietnam for implementation and enhancement of PDCA cycle in road infrastructure asset management. Therefore, as the most important part in this research, intensive study focuses on how to strategically implement road asset management in Vietnam that consists of development a new PMS taking local conditions and advanced data processing techniques into account, optimal policy on data works including collection technology, and initiate PDCA implementation. It is very interesting that the outcomes of our research have been realized fully in reality by the practical process under a tight collaboration scheme between practitioners and academic researchers.

Among seven chapters in this research, **Chapter 1** discusses general introduction regarding the backgrounds, objectives, scope, and expected contributions of the research. The major objective of the study is establishment of a proper road management framework appropriate for Vietnam administration system by assessing technical capability, institutional capability of cross sectional coordination, and decision making process for management plan and budget allocation. The roadmap comprises approaches, programs, projects with activity components that should be conducted follow a concrete schedule.

In **Chapter 2**, pavement deterioration forecasting models, the core of any pavement management system, were discussed with the focus on Markov chain and stochastic deterioration hazard model, and local mixture Markov deterioration hazard model.

An in-depth literature review on hazard model practice in infrastructure asset management in general and pavement asset management in particular has been described in this chapter. The methodology to estimate Markov transition probabilities to forecast the deterioration of road pavements was also discussed intensively. The transition progress among a set of condition states representing the conditions of each pavement section were defined by using exponential hazard models with its parameters are estimated by applying the maximum likelihood method.

Markov chain and stochastic deterioration hazard model provides the solid background and explanation for the local mixture model for benchmarking study that was discussed in the chapter also. The local mixture model is expressed by means of heterogeneity factor ε that exists in each group of pavements. The heterogeneity factor is considered to follow the function of Taylor series. In order to estimate the factor, two steps estimation approach with maximum likelihood estimation method is applied. The local mixture hazard model is considered as an excellent tool for benchmarking study, which is used to support for making the optimum decisions in road management and maintenance in various types such as: search for the best technologies in pavement design, materials, constructions and maintenance through post-evaluation; finding the most suitable application conditions for pavement

technologies that can be traffic intensity, vehicle's axle load, geotechnical conditions, hydraulic and ground water conditions and so forth.

Finally, empirical study applying local mixture hazard model has been researched to identify the differences of pavement deterioration process in 21 different national roads in the northern Vietnam with total length of 2,303 km corresponding to 4,606 lane-kilometers in both directions. The estimated results of exponential hazard models, pavement life expectancies in all major pavement indicators such as IRI, rutting depth, crack ratio and its overall index of MCI gives one comprehensive image for road administrators to make proper evaluation of road conditions, its soundness or quality for taking right actions.

For further introduction of data processing techniques, **Chapter 3** describes one advanced deterioration forecasting model: the compound hidden Markov deterioration model to study the interaction between road surface deterioration and decrease in load bearing capacity of the pavement in case that its surveying time is different. It should be ideal if all information of pavement conditions like pavement surface conditions and its structural capacity are collected in each periodical survey. However, it must be very costly and time consuming especially for structural investigation. Fortunately, the benchmarking evaluation using just pavement surface condition data is able to specify the target sections with bad condition or accelerated deterioration for further structural investigation instead of wide scope implementation for the whole network. That's why in reality it is difficult to simultaneously obtain data on the pavement surface soundness level and its load bearing capacity. But there is no doubt about the mutual interaction between pavement surface conditions and its strength: the better the surface condition, the more durable the pavement and vice versa; the more lower the pavement strength, the more deteriorated the surface and and vice versa. Therefore, it is very crucial and practical to examine such mutual interaction.

This chapter discusses about formulation a compound hidden Markov deterioration model, which demonstratively considers systematic absent data. Additionally, the study also proposes a method that allows the estimation of model parameters with the MCMC (Markov chain Monte Carlo) method based on the results of pavement surface condition and FWD (Falling Weight Deflectometer) surveys, which are not conducted simultaneously. As the final part of the chapter, empirical study not only demonstrates and analyzes the algorithm with missing data but also shows informative and significant results of pavement deterioration forecasting in terms of its surface soundness or strength taking into account the fact of lacking one of these data types. Mutual interaction between two features of road pavement is very clear. Pavement insufficient load bearing capacity takes great effect to the deterioration of its surface that leads to accelerated reduction of the overall level of service. Influence of the load bearing capacity on the road surface soundness level is larger than that of the road surface soundness level on the load bearing capacity.

Compound hidden Markov deterioration model is one of the advanced data processing techniques that is able to take full advantages of available data such as road surface condition and load bearing capacity in case of pavement to provide richer information and results for asset administrators. It is believed that the technique can be expanded for successful application in a variety of fields in asset management.

Chapter 4 presents a new PMS named as "Kyoto model" for pavement management which is consistent with the new international standard ISO55000 for asset management. The Kyoto model proposed in this study adopts methodology that estimates deterioration hazard model by using repair history data, and available datasets of pavement conditions periodically collected through its life cycle.

Implementation of PDCA cycle is so significant because any relevant activities can be optimized to improve pavement management and maintenance continuously that may increase pavement longevity, minimize its life cycle cost (LCC), and strengthen accountability to road users, tax payers and public. In the PDCA cycle, post-evaluation function in the PMS Kyoto model based on accumulated data from "Check" would bring useful information and valuable recommendations to take action ("Act") for enhancement of "Do" or "Plan" as well. Over the time, PDCA cycle would be enhanced also.

Mathematical models are used to develop the PMS Kyoto model with the logical methodology that is explainable for deeply understanding. Being master of the system, any requirement on customization the model to fit to the actual conditions of application can be satisfied. Based on the platform of Kyoto model, it is possible for road administrators in each region, nation to develop their own customized PMS system to support for their PDCA cycle in their infrastructure asset management.

Within the institutional framework under each application condition, functions of PMS system can be different that may vary from simple functions to sophisticated ones. However, in any case, data works must be the first priority for implementation of PMS in particular and PDCA in general. Infrastructure asset management, time-series data should be one of the valuable asset as defined by the international standard for asset management known as ISO55000. And Kyoto model is planned to support and promote for any minor effort of data works.

The deterioration forecasting methodology in Kyoto model is quite sophisticated to makes it work as data-oriented model and extremely support for road management including any initial effort of data preparation. The case study on pavement deterioration in National Highway No.2 in Vietnam demonstrates that with the first effort of the management company in pavement condition survey in 2014 for just 88 kilometers.lane, informative and useful results are made to support them approach to planned and strategic maintenance.

As the master of fact, beside the technical efforts, there must be proper institutional arrangement to define the relevant stakeholders of PMS system and the demarcation among their responsibilities from data preparation, system analysis to system upgrading and expansion.

In **Chapter 5**, efforts and implementation for development a new PMS to customize and make it adapt to the local conditions in Vietnam were discussed. Long experience with big lessons learned regarding application of commercial asset management systems leads to definitive decision to approach to open system as the new direction. Achievement obtained from applying Kyoto model in Vietnam in particular and dissemination of comprehensive knowledge in asset management within the collaboration framework between Japan and Vietnam are the most significant factors for such decision. Among various collaboration programs, annual training courses on road infrastructure asset management jointly provided by Kyoto University, Japan and University of Transport and Communications, Vietnam for years for many Vietnamese engineers, researchers, practitioners play the crucial role.

The final effort for realizing the new PMS in Vietnam is the practical project for capacity enhancement in road maintenance funded by JICA in the form of cooperation scheme that is much different from common technical assistant (TA) projects from other donors in terms of effective technology transferring and knowledge dissemination.

The new PMS has been developed by customizing the Kyoto model as a platform, adopting methodology that estimates deterioration model by using repair history data, and available datasets of pavement conditions periodically collected through its life cycle. The estimated performance curves explain the tendency that pavement deteriorates. Budget planning for different maintenance scenarios will be made based on performance curves and repair policy or flowchart of work repair selection taking into account of priority of candidate repair sections. Benchmarking evaluation is also useful function to identify target groups of pavement sections with fast deterioration speed to formulate plan of detail investigation as well as to verify appropriateness of pavement technological alternatives under certain conditions. Concrete procedures for development the new PMS and detail description on its functions, component modules including operation and computation flow of each module are discussed in the chapter. The description is expected as the good and useful reference for expanding the development of management systems for other facilities from pavement or to other countries especially in Asian region.

Chapter 6 discusses the initial implementation of PDCA cycle on road pavement based on empirical study applying useful tool of the Kyoto model-based PMS that was newly developed.

In this chapter, the main research focuses on improvement of pavement design practices in Vietnam based on post-evaluation of pavement performance using rich database of surveyed

pavement conditions. Situation in the reality of increasing pavement distresses in both the extension and severity is the motivation for systematic researching to find out the main causes for proper solution. Current practices of pavement design have been studied. Surveyed data of pavement condition has been accumulated and integrated to formulate PMS dataset with the special consideration by grouping samples into five different types in term of road sections (normal section, bridge approach section, intersection approach section, bridge deck section, and toll approach section) and two types of road lanes (heavy vehicle lane and light vehicle lane). Benchmarking analysis applying local mixture hazard model shows impressive results that properly reflects pavement deterioration in reality and clarifies improper points in current pavement design practices need being improved. Special consideration should be paid on pavement structure design for critical sections along the road (bridge approach section, intersection approach section, bridge deck section, and toll approach section) and critical lanes of heavy vehicle lane on multiple-lane roads. Moreover, the results also point of the improper point in current pavement design standard in Vietnam. Lacking of performance model for permanent deformation leads to the difficulty in pavement rutting control.

Finally, recommendations for improvement of pavement design standard has been made that consist of gradual transition to optimal applying high performance material such as polymer modified asphalt for pavement taking into account of limited budget source, and permanent deformation model to examine rutting resistance of designed pavement.

In **Chapter 7**, research contents are summarized to briefly emphasize the practical implementation of pavement management system in Vietnam as a case that is worth for reference or dissemination to extend to other fields or other nations especially developing countries. Overall conclusions are presented in the chapter also.

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Abbreviations

A

AADT	Annual Average Daily Traffic
AASHO	The American Association of State Highway Officials
AASHTO	The American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ADB	Asian Development Bank
AMS	Asset Management System
ASTM	The American Society for Testing and Materials

B

BM	Benchmark
BPM	Bituminous Penetrated Macadam
BST	Bituminous Surface Treatment

C

CR	Crack Ratio

D

DB	Database
DBST	Double Bituminous Surface Treatment
DOF	Department of Finance
DOT	Department of Transportation
DRVN	Directorate for Roads of Vietnam

E

ESAL	Equivalent Single Axle Load

F

FWD	Falling Weight Deflectometer

G

GIS	Geographic Information System

H

HCM (in Vietnamese)	Ho Chi Minh Highway
HCM	Highway Cost Model

HDM	Highway Development Management

I

IMU	Inertial Measurement Unit
IRI	International Roughness Index
ISO	

J

JICA	Japan International Cooperation Agency

K

L

LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LOS	Level of Service

M

MCI	Maintenance Control Index
MCMC	Markov chain Monte Carlo
MEPDG	Mechanistic-Empirical Pavement Design Guide
MIT	Massachusetts Institute of Technology
MLIT	Ministry of Land, Infrastructure, Transport and Tourism of Japan
MOF	Ministry of Finance
MOT	Ministry of Transport

N

NA	National Assembly
N/A	Not Available
ND-CP	Government Decree
NEXCO	Nippon Expressway Company Limited
NEXCO RI	NEXCO Research Institute
NH	National Highway

O

ODA	Official Development Assistance
OL	Overlay

P

PBC	Performance-Based Contract
PC	Portland Concrete
PCI	Pavement Condition Index
PDCA	Plan - Do - Check - Act
PDOF	Provincial Department of Finance
PDOT	Provincial Department of Transportation
PIARC	The World Road Association
PMAS	Pavement Management Accounting System
PMoS	Pavement Monitoring System
PMS	Pavement Management System
PRRMC	Provincial Road Repair and Management Company
PSI	Present Serviceability Index
PSR	Present Serviceability Rating

Q

QD-TTg (in Vietnamese)	Decision of the Prime Minister
QH (in Vietnamese)	National Assembly

R

RD	Rut Depth
R/D	Research and Development
RRMB	Regional Road Management Bureau
RRMC	Road Repair and Management Company
RRMU	Regional Road Management Unit

S

SAPI	Special Assistance for Project Implementation
SB	Sub-Bureau of Road Management
SBST	Single Bituminous Surface Treatment
SDCA	Stepwise Directional Customization Approach
SOE	State Owned Enterprise

T

TA	Technical assistant
TBST	Triple Bituminous Surface Treatment
TCDBVN (in Vietnamese)	Directorate for Roads of Vietnam

TCCS (in Vietnamese)	In-house Standard
TCN (in Vietnamese)	Sector Standard
TCVN (in Vietnamese)	National Standard
TRL	Traffic Research Laboratory
TT-BGTVT (in Vietnamese)	Circular of the Ministry of Transport
TV	Traffic Volume

U

UTC	University of Transport and Communications
USA	The United States of America

V

VITRANSS	Vietnam National Transport Strategy Study
VRA	Vietnam Road Administration

W

WB	World Bank

X

Y

Z

CHAPTER 1

Introduction

1.1 GENERAL INTRODUCTION

In order to meet the increasing demand of the socio-economic development, much attention and huge capital had been paid and invested for development of the road infrastructure in Vietnam. Currently, infrastructure construction is rapidly growing in recent years and thus infrastructure stocks are piling up rapidly. It is strongly recommended to take timely actions in order to secure for proper maintenance for infrastructure including formulation strategic plans and their implementation guidelines for infrastructure maintenance at an early stage.

Asset management is crucial for road infrastructure. Road infrastructure shall function efficiently for a long period with proper asset management for constant economic growth of Vietnam. On the other hand, inadequate management causes accelerated deterioration and declining functional capacity of the facilities, which would result in economic loss and unexpected impacts to the society. However road infrastructure asset management had not been fully implemented in Vietnam for years.

There exist various bottlenecks preventing the existing framework of road management from producing expected benefits; such as insufficient quality and quantity of monitoring data, ineffective utilization of installed system, lack of proper system or personnel support for evaluating project performance, insufficient institutional capacity to manage the maintenance work, and poor awareness of communities and decision makers about the significance of conducting right road infrastructure asset management.

In light of the situation, a roadmap for sufficient asset management of road network in Vietnam should be established based upon comprehensive study. The major objective of the study is establishment of a proper road management framework appropriate for Vietnam administration system by assessing technical capability, institutional capability of cross sectional coordination, and decision making process for management plan and budget allocation. The roadmap comprises approaches, programs, projects with activity components that should be conducted follow a concrete schedule.

1.2 RESEARCH BACKGROUNDS AND PROBLEM STATEMENT

Being aware early the significance of management and maintenance the road infrastructure, right after Doimoi (or Renovation) in parallel with the efforts and investment for development and expanding the road network, approach to road asset management systems such as HDM-3, HDM-4, ROSY System, RoadNam, Stripmap, etc,. had been made mostly within framework of donors's technical assistances. One of the first efforts was made for road database formulation for 4,544 km of national highways in the duration from 1994 to 1996. Later on, ROSY System and HDM-3 had been introduced for application since 1998.

After more than one decade of application these systems including deployment of many technical assistances with the huge total investment of hundreds million USD for software procurement, license fees, technology transfer, trainings, data collection and surveys, and so forth. In the International Seminar of Road Asset Management held in Hanoi in October 2010, it was confirmed by the road administrator that the deployment and application of these systems in Vietnam for long time is unsuccessful.

Such unsuccessful consequence could be understood in following features:

- Due to the lacking of customization to local conditions in road management and maintenance, the systems are not suitable for application in Vietnam. It can be explained that these systems had been developed for general conditions of common utilization without or with insufficient consideration of specified characteristics that may vary broadly country by country. As in the example on point of view on cost related to road infrastructure, there exist big different between developed and developing countries. While administrator's cost for road maintenance and management is the main attention in developing countries, the rich countries usually pay attention to not only administrator's cost but also road users' cost and social cost as well. The consideration may leads to the difference in system development and data preparation also.
- It is understood that keep using these systems shall increase the burden on budget for system and license updating, trainings and technology transferring because these systems are commercial softwares and black-box also. Huge budget was allocated for data collection but utilization of the data is not intensive. It seems that for these systems, history data does not make sense.
- The goal of enhancement capacity for human source in the road management and maintenance field had not been achieved. Many trainings and technology transferring had been made but focused on teaching how to operate the systems rather than fundamental knowledge on strategic management and maintenance including the concept of PDCA cycle on asset management.

Lacking of useful tools for decision making, road management in Vietnam has remained passive. Decisions had been made by judgement based on experience without sufficient systematic and quantitative bases. Mid-term and long-term road maintenance plans had been formulated but the argument and methodology are convincing to negotiate with competent authorities such as Ministry of Finance or the Government in budget allocation. Therefore, the situation of lacking budget for road maintenance has been remaining seriously for years that creates a barrier for approaching to preventive or proactive maintenance. Optimization in road asset management is hardly expected. The situation becomes more critical because PDCA cycle for road asset management had not been formulated or introduced even the concept. Road management is just a simple one-way flowchart. Many pavement distresses in the site in reality can be recognized as the causes of previous phases in construction, structure or material mix design but the scheme of feedback for improvement is very weak.

Naturally, this fact demands for changing to more suitable and sustainable approach of road asset management in Vietnam. PDCA cycle must be implemented. That requires for one good tool of PMS system which should be newly developed for specific conditions and practical requirements in Vietnam with rich involvement of Vietnamese engineers. Sustainable roadmap of system development including upgrading must be specified.

However, due to the natural practices of using ready-made systems or softwares for years in combination of the lacking of firm and essential knowledge and methodologies in road infrastructure asset management, it is so challenging to realize such about direction. Being highway engineer, I had studied subjects and accumulated knowledge of road management and maintenance in universities in Vietnam. In this knowledge, it is impossible to find out systematic approach for road management such as strategic maintenance planning, deterioration forecasting, life cycle analysis, systematic selection of repair work, and so forth. Since 1999, as university lecturer in highway engineering, I had paid much attention and time for deepen my study in the field in order to improve my lectures for students. But at that time, my approach to more logical methodology based on domestic materials including textbooks is at a standstill also. It is impossible for me to find any textbooks, lectures in Vietnamese in highway engineering that introduce new methodologies in strategic road asset management. Therefore, I had planned the determination to pursue this field for researching to improve my knowledge especially to disseminate such knowledge to our many students who will be the young generations of engineers and practitioners for the goal of enhancement road asset management in Vietnam to maintain the road infrastructure asset in particular and national infrastructure assets in general for sustainable transferring to next generations. This is the utmost motivation for me to decide the research topic of this PhD dissertation.

Since 2010 in Vietnam, it has been obvious to recognize attention of the Government on institutional renovation in road management. Central road management administrator, Vietnam Road Administration (VRA) was upgraded to higher level of Directorate for Roads of Vietnam

(DRVN) for enhancement of their capacity and state management function on roads as well. Intensive reviews of current practices on road management had been conducted for fact finding of identified problems and shortcomings in this field for formulation a comprehensive renovation program in road asset management. During implementation the program, beside the internal efforts from Vietnam, there had been many positive and effective supports from Japanese Government and tremendous assistance of individual experts and organizations. And it is so lucky for me that during the long duration of such renovation implementation, I have had change to get direct and full involvement and take my personal contribution especially in practical implementation of pavement management system in Vietnam by developing new system, training provision and support for execution of PDCA cycle in road asset management in Vietnam.

1.3 OBJECTIVES OF RESEARCH

In this research, the objectives have been specified that consist of three concrete items as follows:

- The first priority in this research is to study thoroughly the current practices of road management and maintenance in Vietnam to find out the problems especially the weakness of supporting tools for decision making on strategic pavement planning. Applying Pavement management system Kyoto model using available data to forecast pavement deterioration for national roads in Vietnam is requested to examine the feasibility of the model under local conditions. Recommendation to road administrator on changing the approach to develop new customized system instead of using black-box systems will be made based on these initial results.
- System components of new PMS system and roadmap for development at the initial stage and upgrading at the next stages are specified to make recommendation for road administrator on proper preparation and arrangement especially on human resource and institutional issues.
- To introduce PDCA cycle on road management and maintenance with the initial effort in fact finding of pavement condition and its deterioration in “Check” to make proper and necessary feedbacks for improvement of “Plan” in previous phase. Large scale of Kyoto model application using datasets of surveyed pavement conditions to examine its actual condition and progression of pavement deterioration by Markov hazard model and Local mixture Markov hazard model is performed.

1.4 SCOPE OF RESEARCH

Outlines of scopes are given as follows

- In Chapter 2, focus is on benchmarking evaluation by local mixture Markov hazard model. Estimation approach for heterogeneity factors for local groups in the mixture and a scheme for benchmarking application is the main discussion. Deterioration of pavements of highways in the northern Vietnam has been analyzed in the empirical study.
- Employing compound hidden Markov deterioration model to examine the mutual action between road surface deterioration and decrease in load bearing capacity of the pavements in the practical case that it is difficult to simultaneously obtain both datasets on surface soundness and load bearing capacity of road pavement. Empirical study was conducted on pavements on the expressways managed by Nippon Expressway Company Limited (NEXCO). This study is described in Chapter 3.
- Chapter 4 describes a new PMS named as "Kyoto model" for pavement management which is a "Data Oriented Road Management System" to support PDCA cycle of road maintenance work. Kyoto model manages information acquired during the on-site maintenance work, and provide with solutions by analyzing the field data, based on deterioration performance evaluation and Benchmarking analysis.
- Chapter 5 presents the practical project in Vietnam for capacity enhancement in road maintenance with the focus on development of a new pavement management system customized from Kyoto model to make it adapt to the local conditions in Vietnam. Beside technical issues, institutional strengthening and human capacity improvement are strongly emphasized.
- The first approach for implementation of PDCA cycle is discussed in Chapter 6. Based on road surveyed data and the new pavement management system, empirical study on different schemes of pavement deterioration has been conducted. Recommendations on pavement design and management were proposed based upon fact findings from the study.
- Conclusions and recommendations on models and empirical studies are given at every last section of respective chapters. Brief summary and overall conclusion are shown in Chapter 7.

1.5 EXPECTED CONTRIBUTION

It is highly expected that, during implementation and after completing the research, the paper and the accumulated knowledge of this research will make contribution to some extend as follows:

- To promote for the change of approach in road management and maintenance from passive, ineffective manner to proactive, strategic and systematic scheme.
- To support for realization of the development new customized pavement management system fitting to local conditions in both physical and institutional in Vietnam. Around the final stage of this research, by all efforts from relevant parties especially the proper awareness of road administrators, great supports from Japanese government, researchers and experts, such dream has become true. New customized and "touchable" PMS system has been developed with my minor contribution.
- Description of the practical project in Chapter 5 shows the general roadmap and implementation to develop PMS system. It is expected that the case study can be expanded to infrastructures other than pavement such as bridges, tunnels, airports, etc., in Vietnam or other Asian countries.
- In Chapter 6, application of Markov hazard model and Local mixture Markov hazard model in the empirical study of pavement deterioration in the Northern area in Vietnam has pointed out the big gaps of pavement life spans for different road sections in longitudinal direction and vehicle lanes in transversal cross-section. It is expected that such analysis results will make useful feedback for improvement of practices on pavement engineering especially on pavement structure design. Recommendations of focused points for enhancement the current design standard of flexible pavements has been pointed out.
- Knowledge accumulated in this research on infrastructure asset management in general and sophisticated deterioration forecasting models is strongly hoped to bring in innovative academic contributions in Vietnam. Some useful textbooks are also expected for publishing to disseminate the knowledge to many road practitioners, engineers and young generations of students who is studying in universities. Consequently, research and development capacity in this field will be improved based on the big transition of human capacity enhancement.
- The scheme of collaboration in this research on practical issues in Vietnam can be regarded as one of the typical example for effective collaboration between Japan and Vietnam. Expanding this scheme is expected to bring more fruitful outcomes from a variety of fields of collaboration between the two countries.

CHAPTER 2

Benchmarking Evaluation by Local Mixture Markov Hazard Model with Pavement Application

2.1 GENERAL INTRODUCTION

Infrastructure asset management in developing nations is about at the outset of development like that of developed nations in the beginning of economic boom period several decades ago. Thanks to the earlier development in various types of infrastructure technologies in developed nations like the America, Japan, and other European countries including introduction and dissemination, developing countries can take a great advantage in view of technology and management to apply in their nations. As a rule of thumb, in order to introduce foreign technologies into developing nations, technologies need to be verified its adaptability and conformity with local conditions. And customization with taking all local conditions into account must be indispensable. However, the practice of verification has not yet been seriously or fully considered in actual implementation. Various types of technologies in construction industry have been applied but lacking of quantitative and scientific evaluation.

Selection for the best technology, approach or making the right decision in the infrastructure management and maintenance is thereby, having a close link to the methodology of benchmarking, which provides a managerial approach for finding out the best practice. Evidently, the core part of benchmarking study in the field of pavement management or in PMS is the application of hazard model. Because, hazard model is indispensable in estimating the deterioration of a road network, which becomes the most important key performance indicator for benchmarking.

In the network level of PMS, there are many groups of road, which are often categorized by the different criteria such as pavement technology, loading or traffic intensity conditions, natural conditions, operation and maintenance conditions, and so forth. When applying conventional hazard models, monitoring data of the entire network is considered as a representative database used to predict the deterioration of the entire network system. This practice has its limitation that it is difficult to apply for estimating the deterioration of

respective groups in the network. Especially, under the requirement of benchmarking, which aims to compare the deterioration of individual group of road in the network.

In regard to the development of deterioration forecasting model, in recent years, the application of Markov chain model has been one of the major innovations. Markov hazard model helps users to predict hazard rates, life expectancies, and deterioration curves of infrastructure system such as bridges and highways given the historical observed condition states and characteristic variables concerning various environment impacts such as traffic volume, weather, temperature, axle load, etc. The application of Markov chain model has gained its high recognition for its flexibility of modeling and high operational capacity [1], [2]. However, the study of Markov hazard model to tackle the problem of inhomogeneous sampling population is not numerously documented.

This chapter presents an analytical methodology to obtain the heterogeneity factor of individual pavement group based on the local mixture hazard model, and further explore a benchmarking application in order to make the best decisions in road management and maintenance. The following section gives an overview of research background with focus on Markov chain model and mixture hazard model. Section 2.3 presents the fundamentals of Markov chain and stochastic deterioration hazard model [2], to which, this study herein builds on. Section 2.4 details the mathematic formulation of local mixture hazard model and its estimation approach. An empirical study on the national road system of Vietnam is discussed in Section 2.5. Finally, section 2.6 summarizes the contributions of the research and further includes a discussion for future research.

2.2 BACKGROUND

In order to study the chance process, there have been variety of approaches and probability theory is the well-known approach with very wide application in many fields so far. Independent trials processes are the basis of classical probability theory with the two principal theorems of: (1) the Law of Large Numbers, and (2) the Central Limit Theorem. In the case that a certain sequence of chance experiments creates an independent trials process, the possible outcomes for each try are the same with each other and occur with the same probability also. It is also understood that there is no influence from the outcomes of the previous tries to the outcomes of the next ones.

In difference with the classical probability theory, there have been deep studies and researches in modern probability theory investigates chance processes for which the knowledge of previous outcomes influences forecasting results for future.

In 1907, A. A. Markov began his study on an important new type of chance or stochastic transition process. In this process, the outcome of a given experiment also affect the outcome

in the next ones. And this process is called a Markov chain. Markov chain models have been widely applied in the practice of deterioration-forecasting in infrastructure management [3], [4]. In Markov chain models, the healthy status or performance of an infrastructure component is described in discrete condition states, which are defined by means of single performance indicators or an overall index. The values of indicators are measured by monitoring and visual inspection or survey. For example, in the case of pavement management system, the condition states include the extend of several pavement individual distresses such as rutting and cracking, or some indices, such as the Pavement Condition Index (PCI) [5]. The deterioration of an infrastructure component is portrayed as a transition process of condition states along the duration.

2.3 MARKOV CHAIN AND STOCHASTIC DETERIORATION HAZARD MODEL

2.3.1 Condition State and Periodical Inspection

It is natural that right after putting the road into operation, pavement deterioration initiates at once as the accumulation negative effects caused by traffic intensity, critical weather conditions and so forth. Such deterioration makes reduction in pavement performance or level of service much or less depending on various factors of accumulated loads, pavement age, construction quality and applied technologies, etc.,. When its performance curve approaches to the critical level, there must be implementation of maintenance work or reconstruction in the worst case.

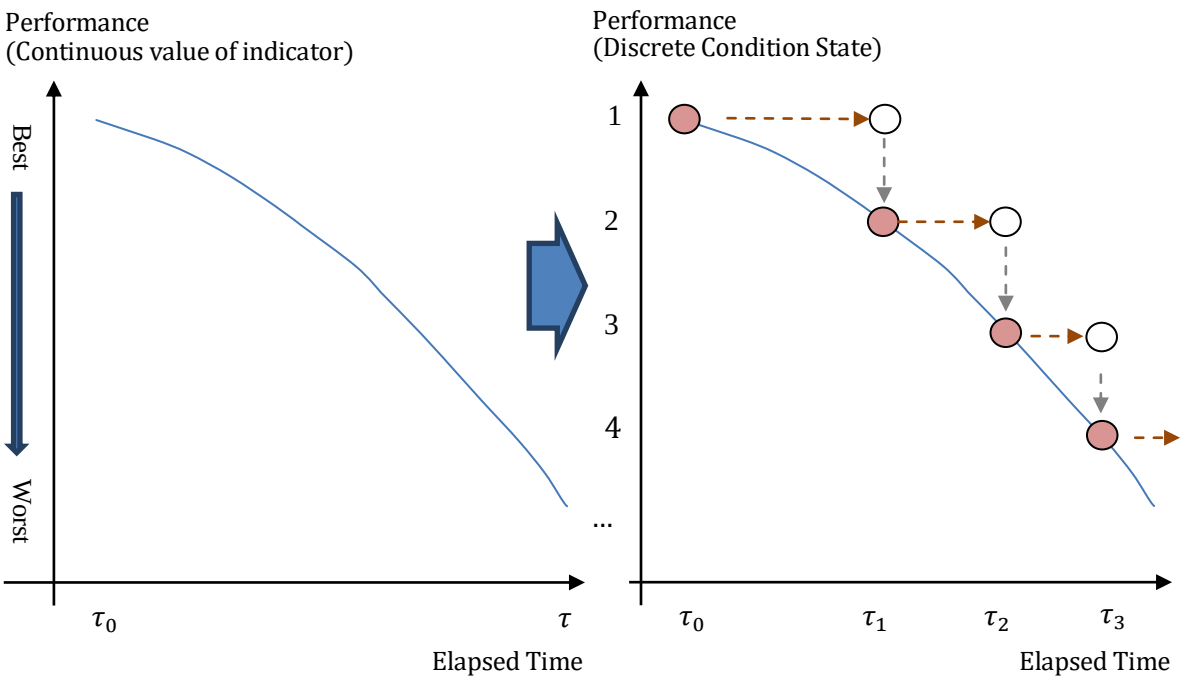
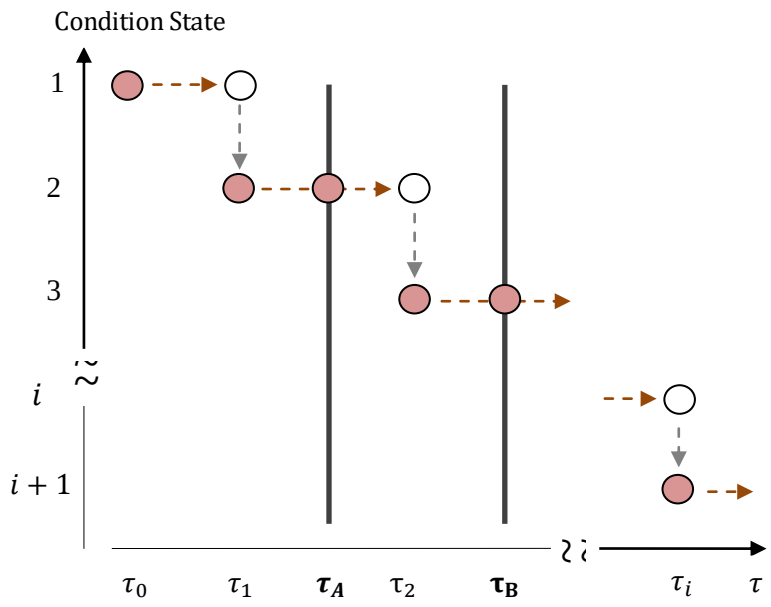


Figure 2.1 Pavement Deterioration Process and Conversion to Discrete Condition States

In order to clarify pavement deterioration, it is necessary for implementation of pavement condition surveys or inspections. The more number of inspection times or in case of continuous inspection, the better performance curve can be drawn to show continuously absolute value of pavement indicators to satisfy requirement on pavement monitoring. However, such approach will burden the demand of data collection which is agreed to be costly and time consuming especially in case of the entire road network. Moreover, beside monitoring, it is necessary to make maintenance plans which require for the image of future pavement condition from predicted data. Therefore, one different approach of applying relative value of a finite number of discrete ranks or condition states instead of continuously absolute value had been selected to characterize pavement condition. Figure 2.1 shows pavement deterioration process and conversion to discrete condition states.



Note) In this illustration, pavement deterioration process is expressed in terms of calendar time $\tau_1, \tau_2, \ldots, \tau_i$, and condition states of the sections which increases in unitary units corresponding to the decrease of pavement conditions from the best to the worst.

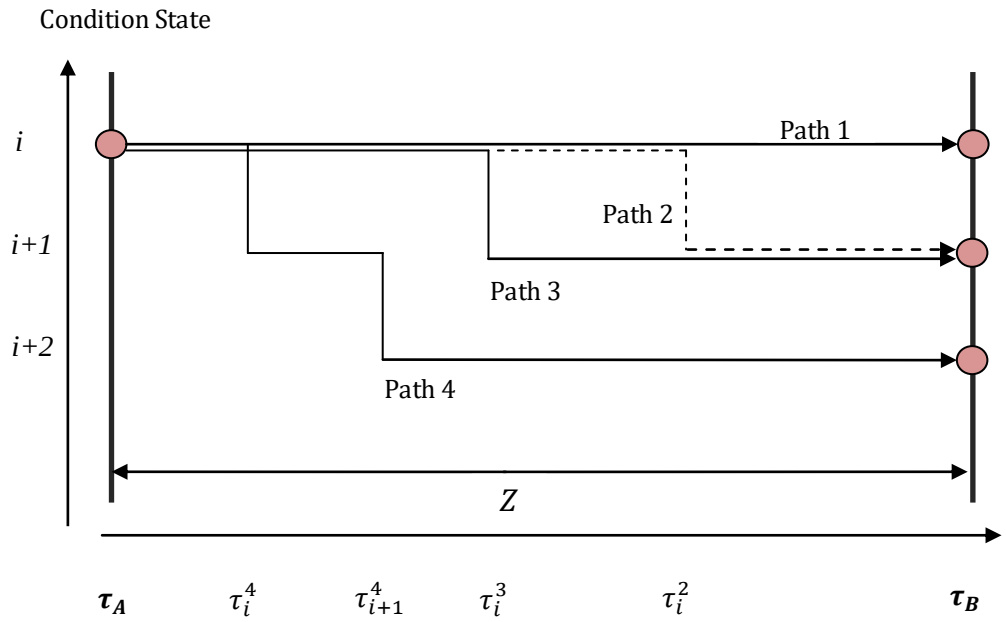
Source: (Tsuda et al, 2006)

Figure 2.2 Pavement Deterioration Process in Timely Transition of Condition States

Figure 2.2 shows the transition process of pavement condition during its life cycle. In this figure, τ represents real calendar time, the condition state of a pavement is defined by a rank J representing a state variable $(i = 1, \ldots, J)$. Right at the time of construction completion to make the road be ready for opening to the public at time τ_0, pavements are at the best condition state which is rated at rank or condition state $i = 1$. Pavement deterioration initiates right after the calender time τ_0 which makes the increasing of condition state i afterward. A value of $i = J$ explains the case that a pavement section has reached its service limit or

absorbing state in Markovian theory. In the figure, for each discrete calendar time τ_i $(i = 1, \dots, J-1)$ on the horizontal time-axis, the corresponding condition state increases from i to $i+1$. Therefore, hereinafter τ_i is referred as the time of occurrence the transition of pavement condition from a condition state i to $i+1$.

Information relating to the deterioration process of pavement can be obtained through a certain number of periodical surveys or inspections. Figure 2.2 also shows two periodical pavement condition surveys at times τ_{A} and τ_{B} on the time-axis for consideration and further study on transition process of pavement condition between these two calender times or within interval time between such two inspections. Detail of the transition process is expressed in Figure 2.3



Source: (Tsuda et al, 2006)

Figure 2.3 Transition patterns of Condition State between two Inspections

It is supposed that at time τ_{A}, the condition state determined by survey is $(i = 1, \dots, J-1)$. The deterioration progress in future times is uncertain. Among the infinite set of possible scenarios describing the deterioration process, only one path is finally realized as the result of next survey or inspection. In this illustration, the deterioration transition of pavement condition is expressed in four possible options of transition paths. Path 1 displays the case of no transition in the condition state i during the inspection interval. In paths 2 and 3 the condition state has increased to one upper condition state at the calendar times τ_i^2 and τ_i^3 respectively. The condition state in these two paths observed at time τ_B becomes $i+1$. In the inspection scheme shown in Figure 2.3, it is impossible to determine the point times τ_i^2 and τ_i^3 at which the condition state changes from i to $i+1$. In path 4, the condition state has increased one state at each time τ_i^4 and τ_{i+1}^4 to make the change of two ranks in total observed at τ_{B} in state $i+2$.

Pavement condition or its condition states are observable at the inspection time or survey that expresses the transition. However, it is impossible to determine information of the times in which those transitions occur.

2.3.2 Markov Transition Probability

In order to estimate a model to forecast the deterioration of pavement, it is necessary to accumulate time series data on its condition states. Given the condition state obtained in the survey at time τ_{A} as shown in Figure 2.3, there are some possibilities of condition states at time τ_B in the future which can be characterized by corresponding probabilities. Therefore, to predict uncertain transition process of the condition states of pavement, stochastic approach is applied with the introduction of Markov transition probability to represent the uncertain transition of pavement condition states within two point times or within one transition step. Markov transition probabilities are defined and applied to predict pavement deterioration using periodical inspection or survey.

State variable $h(\tau_{\mathrm{i}})$ is defined to express the observed condition state of the component at time τ_{i}. Figure 2.3 shows the condition state observed at time τ_{A} is i, so the state variable $h(\tau_{\mathrm{A}}) = i$. A Markov transition probability, given a condition state $h(\tau_{\mathrm{A}}) = i$ observed at time τ_{A}, defines the probability that the condition state at the future time (τ_B for example) will change to $h(\tau_B) = j$. That is

$$Prob[h(\tau_B) = j | h(\tau_A) = i] = \pi_{ij} \tag{2.1}$$

For a set of condition states $(i = 1, \dots, J)$, the matrix of Markov transition probabilities can be defined by using the transition probabilities between each pair of condition states (i, j) among the set as follow

$$\boldsymbol{\Pi} = \begin{pmatrix} \pi_{11} & \cdots & \pi_{1J} \\ \vdots & \ddots & \vdots \\ 0 & \cdots & \pi_{JJ} \end{pmatrix} \tag{2.2}$$

The Markov transition probability π_{ij} shown at the Eq. 2.1 explains the transition probability between the condition states $h(\tau_{\mathrm{A}}) = i$ and $h(\tau_B) = j$ at two given times τ_{A} and τ_B or time interval Z. Probability π_{ij} depends on not only condition states i, j but also on time interval Z. Since deterioration continues as long as no repair is executed that make all probabilities π_{ij} $(j < i)$ become zero, $\pi_{ij} = 0$ $(i > j)$, because pavement condition can not be better without any maintenance. From the definition of transition probability, $\sum_{j=1}^{J} \pi_{ij} = 1$. As properties of probability and nature of pavement deterioration, all following conditions must be satisfied.

$$\left.\begin{array}{l} \pi_{ij} \geq 0 \\ \pi_{ij} = 0 \ \ (when\ i > j) \\ \Sigma_{j=1}^{J} \pi_{ij} = 1 \end{array}\right\} \tag{2.3}$$

The most critical level of deterioration is expressed by the condition state J in the set of condition states, which remains as an absorbing state in the Markov chain. As the properties of Markov chain $\pi_{JJ} = 1$. In reality, absorbing in the condition state J shows the end of pavement service life which requires for taking action of pavement improvement or reconstruction totally in the worst case at the end of its life cycle.

Markov transition probabilities are defined independently from previous history of deterioration. As shown in Figure 2.3, the condition state at the inspection time τ_{A} is i but it is unobservable to know the time in which condition state changed from $i-1$ to i. In one Markov chain model, as the property of Markov chain, it is regarded that the transition probability between the inspection times τ_{A} and τ_B is dependent on the condition state at time τ_{A} or current situation only.

Markov chain starts in a state chosen by a probability distribution on the set of defined condition states $(i = 1, \ldots, J)$, which is called a initial probability vector. A probability vector is a row vector of J components whose entries are non-negative and sum to 1. The ith component of initial probability vector u describes the probability that the chain starts in the condition state i.

In the long-term behavior of a Markov chain, based on probability transition matrix and initial probability vector, it is possible to predict distribution of condition states in the future as the general theorem of Markov chain. Let $\boldsymbol{\Pi}$ be the probability transition matrix of a Markov chain, and let u be the initial probability vector which represents the starting distribution. Then the probability that the chain is in the condition state i after n steps is the ith entry in the vector $u^{(n)}$

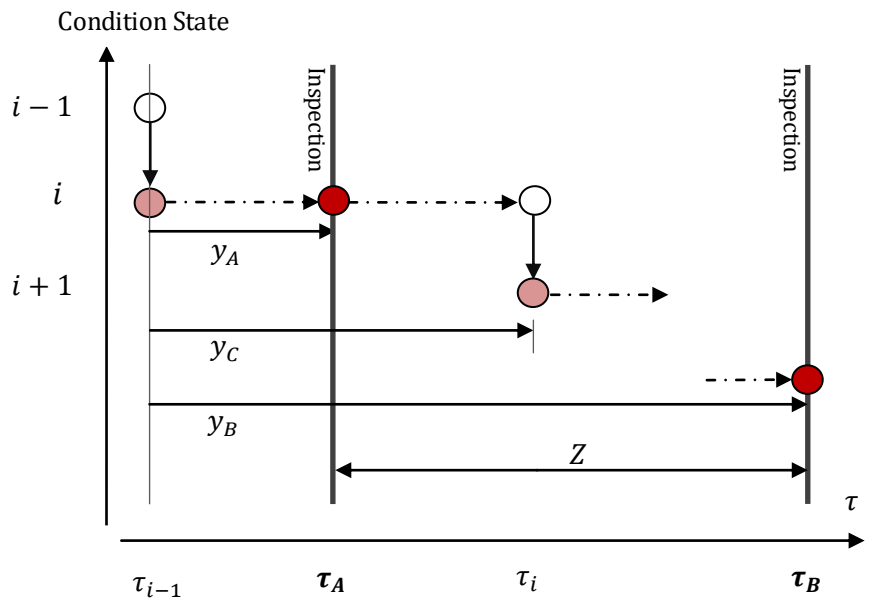
$$u^{(n)} = u\Pi^{n} \tag{2.4}$$

In reality, for system of huge samples like pavement sections on road network, time series data based on periodical inspections can be available but it is quite rare for the case of full availability of all time-series data especially in developing countries. Therefore, application of Markov chain theory and Markov chain model have been increased widely in the field of infrastructure asset management to clarify its deterioration process especially at network level.

2.3.3 Determination of Markov Transition Probabilities

2.3.3.1 Formulation of Hazard Model

The Markov transition probability can be defined by using hazard model characterizing the deterioration process of an individual pavement section. The information obtained by periodical inspection contains not only data of condition state of an individual pavement section, but also disaggregate data specific to the section such as its structural characteristics or the pavement usage conditions and so forth. The inspection intervals may also differ from one pavement section to another or from interval to interval. In order to estimate Markov transition probabilities based on such a variety of data, it is desirable to develop an estimation methodology that can consider specific characteristics of each individual section. Tsuda et al (2006) proposed the methodology to determine Markov transition probabilities based on results of estimated hazard model using periodical inspection data. Markov transition probabilities determined by means of hazard models are referred as disaggregate Markov transition probabilities. The methodology to obtain the average transition probability for the entire pavement sections is later explained in Section 2.3.4.4.



Note)In the case the condition state changes from $i-1$ to i at the calendar time τ_{i-1}, the inspections carried out at times τ_A and τ_B will also correspond to the points in time y_A and y_B when using τ_{i-1} as the time origin. The figure illustrates one sample deterioration path in which the condition state has increased in one rank to y_C in the interval time $\tau_{i-1} - y_C$. However in reality, observations at time τ_{i-1} are not possible in the periodical inspection scheme, so there is no way to obtain observation at y_A, y_B and y_C. Nevertheless, it is possible to use the information contained in $z = y_C - y_A \in [0, Z]$.

Source: (Tsuda et al, 2006)

Figure 2.4 Model of Pavement Deterioration Process

The deterioration process of road pavement illustrated in Figure 2.4 shows the change of its condition state at the calendar time τ_{i-1} from $i-1$ to i. Thus, the calendar time τ_{i-1} is set to be the origin $y_i = 0$ of the time-axis. The time represented by the sample time-axis is referred from now as a 'time point', and differs from 'time' on the calendar time-axis. The times τ_{A} and τ_B in the figure mean the time points y_A and y_B respectively on the sample axis. With such definition of time point, it can be clear that $y_A = \tau_A - \tau_{i-1}$, $y_B = \tau_B - \tau_{i-1}$.

Information on the condition state i at the beginning of the calendar time τ_{i-1} cannot be obtained in a periodical inspection scheme. Therefore, time points y_A and y_B on the sample time-axis cannot be obtained. For simplicity in description, it is assumed that the information at the time points is known in order to develop the model, in spite of this assumption is not necessarily essential. The following paragraph discusses that even in the case of there is no information at time points y_A and y_B, an exponential hazard model can be estimated.

In the case the condition state of one pavement section at time τ_i or time point y_C is assumed to increase from i to $i+1$ during pavement deterioration process, the period length in which the condition state keeps remaining at i is represented by $\xi_{\mathrm{i}} = \tau_i - \tau_{i-1} = y_C$. In which ξ_{i} is referred as the life expectancy of a condition state i. And the life expectancy of condition state i is assumed to be a stochastic variable with probability density function $f_i(\xi_i)$ and distribution function $F_i(\xi_{\mathrm{i}})$. Stochastic variable ξ_{i} is defined in the domain $[0, \infty]$. Following is the definition of distribution function

$$F_i(y_i) = \int_0^{y_i} f_i(\xi_i) d\xi_i \tag{2.5}$$

The distribution function $F_i(y_i)$ represents the cumulative probability of the transition in the condition state from i to $i+1$ under the setting of condition state i at the initial time point $y_i = 0$ or calendar time τ_{A}. The time interval measured along the sample time-axis until the time point y_i is $\tau_{i-1} + y_i$. By introduction of the cumulative probability $F_i(y_i)$, the probability $\tilde{F}_i(y_i)$ of a transition in the condition state i during the time points interval $y_i = 0$ to y_i in the domain $[0, \infty]$ can be defined by $\tilde{F}_i(y_i)$:

$$Prob\{\xi_i \geq y_i\} = \tilde{F}_i(y_i) = 1 - F_i(y_i) \tag{2.6}$$

The conditional probability that the condition state of one pavement section at time y_i advances from i to $i+1$ during the time interval $[y_i, y_i + \Delta y_i]$ is defined in following equation.

$$\lambda_i(y_i)\Delta y_i = \frac{f_i(y_i)\Delta y_i}{\tilde{F}_i(y_i)} \tag{2.7}$$

where the probability density $\lambda_i(y_i)$ is referred as the hazard function.

2.3.3.2 Exponential Hazard Model

In this part, it is assumed that the deterioration of a pavement section satisfies Markov property, and the hazard function is independent of the time y_i on the sample time-axis. That is, for a fixed value of $\theta_i > 0$, we have:

$$\lambda_i(y_i) = \theta_i \tag{2.8}$$

By using the exponential hazard function in Eq. 2.8, it is possible to represent a deterioration process of a pavement section that satisfies the Markov property *(independence from the past history)*. In addition, it is also assumed the difference of hazard functions for different condition states that shows $\theta_i \neq \theta_j \ for \ i \neq j$. By differentiating both sides of the equation 2.6 with respect to y_i

$$\frac{d\tilde{F}_i(y_i)}{dy_i} = -f_i(y_i) \tag{2.9}$$

Equation (2.7) then becomes

$$\lambda_i(y_i) = \frac{f_i(y_i)}{\tilde{F}_i(y_i)} = -\frac{\frac{d\tilde{F}_i(y_i)}{dy_i}}{\tilde{F}_i(y_i)} = \frac{d}{dy_i}\left(-log\,\tilde{F}_i(y_i)\right) \tag{2.10}$$

Take into consideration that $\tilde{F}_i(0) = 1 - F_i(0) = 1$ and by integrating equation (2.10)

$$\int_0^{y_i} \lambda_i(u)du = [-log\,\tilde{F}_i(u)]_0^{y_i} = -\,log\,\tilde{F}_i(y_i) \tag{2.11}$$

Using the hazard function $\lambda_i(y_i) = \theta_i$, the probability $\tilde{F}_i(y_i)$ that the life expectancy of the condition state i becomes longer than y_i is expressed by

$$\tilde{F}_i(y_i) = exp\left[-\int_0^{y_i} \lambda_i\,(u)du\right] = exp(-\theta_i y_i) \tag{2.12}$$

Equation (2.12) is an exponential hazard model. According to equation (2.9), the probability density function $f_i(\xi_i)$ of the life expectancy of the condition state i is

$$f_i(\xi_i) = \theta_i\, exp(-\theta_i \xi_i) \tag{2.13}$$

From now, let's consider that the condition state has changed to i at the time τ_{i-1}, and remains constant until the inspection or survey time τ_{A}. It means that the condition state obtained at the inspection time τ_{A} is i, no change in pavement condition state in comparison with the condition at the time τ_{i-1}. In such case, at the time point y_{A} on a sample time-axis,

the condition state is also i. The probability that the condition state i remains constant in a subsequent time z_i (≥ 0) measured from the time point y_{A} is then defined as following equation

$$\tilde{F}_i(y_A + z_i | \xi_i \geq y_A) = Prob\{\xi_i \geq y_A + z_i | \xi_i \geq y_A\} \tag{2.14}$$

Dividing both sides of equation (2.14) by the probability $\tilde{F}_i(y_i)$ showing in equation (2.6) comes to the result as follow

$$\frac{Prob\{\xi_i \geq y_A + z_i\}}{Prob\{\xi_i \geq y_A\}} = \frac{\tilde{F}_i(y_A + z_i)}{\tilde{F}_i(y_A)} \tag{2.15}$$

By applying equation (2.12), the right side of equation (2.15) becomes:

$$\frac{\tilde{F}_i(y_A + z_i)}{\tilde{F}_i(y_A)} = \frac{exp\,\{-\theta_i(y_A + z_i)\}}{exp\,(-\,\theta_i\, y_A)} = exp(-\theta_i\, z_i) \tag{2.16}$$

For expanding the application, for a condition state i obtained by periodical inspection or survey at time point y_{A} or calender time τ_{A}, the probability that the same condition state will be observed by the subsequent inspection time $y_B = y_A + Z$ is

$$Prob[h(y_B) = i | h(y_A) = i] = exp(-\theta_i Z) \tag{2.17}$$

where Z is the interval between two inspection times as shown in Figure 2.4. The probability $\mathrm{Prob}[h(y_B) = i | h(y_A) = i]$ is the Markov transition probability π_{ii} as mentioned in equation (2.1). Obviously, if the exponential hazard function is applied, the Markov transition probability π_{ii} is dependent only on the hazard rate θ_i and the inspection interval Z. In more significant point, without using deterministic information on the time points y_A and y_B as shown in Figure 2.4, it is totally possible to estimate transition probabilities for formulation of Markov transition probability matrix. Under the sense, it must be understood that periodical inspections or surveys that support formulation of time-series data are the most important issue in forecasting pavement deterioration.

2.3.3.3 Determination of Markov Transition Probabilities

From the above description, transition probability π_{ii} for one case of pavement deterioration that its condition state remains for duration can be obtained to specify the life expectancy of the condition state i. Figure 2.4 also shows the case that the pavement condition state at the inspection or survey time points y_A and y_B changes from i to $i+1$ due to pavement deterioration. Using an exponential hazard function, the probability of such transition can be

obtained. It is clear that such transition of pavement condition state can occur if three following conditions are satisfied:

(1) the condition state i remains constant between a time point y_A to a time point $s_i = y_A + z_i$ for one additional time $z_i, (z_i \in [0, Z])$.
(2) the condition state transits from i to $i+1$ at the time point $y_A + z_i$ and
(3) the new condition state $i+1$ remains constant between the time interval $y_A + z_i,\ y_B$.

Even though the exact time at which the pavement condition state changes from i to $i+1$ cannot be traced by periodical inspection or survey, it can be accepted for a temporary assumption that the transition occurs at the time point $(y_A + \bar{z}_i) \in [y_A, y_B]$. Given the condition state i at the inspection time point y_A or calendar time τ_A that remains constant until the time point $(y_A + \bar{z}_i)$, the probability density that at this time point the condition state transits to $i+1$ is

$$g_i(\bar{z}_i | \xi_i \geq y_A) = \frac{f_i(\bar{z}_i + y_A)}{\tilde{F}_i(y_A)} = \frac{\theta_i exp\,\{-\theta_i(\bar{z}_i + y_A)\}}{exp\{-\theta_i y_A\}} = \theta_i\, exp(-\theta_i \bar{z}_i) \quad (2.18)$$

Satisfying the above conditions, the conditional probability density that the condition state observed at the inspection time point y_B or calendar time τ_B is $i+1$ becomes:

$$q_{i+1}(\bar{z}_i | \xi_i \geq y_A) = g_i(\bar{z}_i | \xi_i \geq y_A).\tilde{F}_{i+1}(y_B - \bar{z}_i - y_A) \quad (2.19)$$

$$= \theta_i exp(-\theta_i \bar{z}_i)\, exp\{-\theta_{i+1}(Z - z_i)\}$$

$$= \theta_i exp(-\theta_{i+1} Z)\, exp\{-(\theta_i - \theta_{i+1})\bar{z}_i\}$$

Regarding to the above description, it is obvious that fixed value of $\bar{s}_i = y_A + \bar{z}_i$ is applied. Therefore, $\bar{z}_i$ may vary in the range $[0, Z]$ because life expectancy ξ_i of the condition state i is a stochastic variable. The Markov transition probability that the condition state changes from i to $i+1$ in the duration between two time points y_A and y_B becomes

$$\pi_{ii+1} = Prob[h(y_B) = i+1 | h(y_A) = i] = \int_0^Z q_{i+1}\left(z_i | \xi_i \geq y_A\right) dz_i \quad (2.20)$$

$$= \int_0^Z \theta_i exp\,(-\theta_{i+1} Z)\, exp\{-(\theta_i - \theta_{i+1}) z_i\} dz_i$$

$$= \frac{\theta_i}{\theta_i - \theta_{i+1}}\,\{-exp(-\theta_i Z) + exp(-\theta_{i+1} Z)\}$$

where $\pi_{ii+1} > 0$ is indifferent to the relative size between hazard functions θ_i and θ_j . The assumption $\theta_i \neq \theta_{i+1}$ implies $1 > \pi_{ii+1}$. As these characteristics are trivial in the derivation process of equation (2.20), the verification is omitted.

Two cases of pavement deterioration or condition state transition that are expressed by probabilities π_{ii} and π_{ii+1} have been discussed so far. From now, the description focuses on the more general case of pavement transition or condition state between two inspection times or surveys changes from i to j in which $j \geq i+2$. The distribution function and the probability density function of one duration in which a condition sate j remains constant is denoted as $F_i(y_j)$ and $f_i(y_j)$. The hazard function in this case is denoted by $\lambda_j(y_j) = \theta_j$.

The process of transition of the condition state from i to j $(j \geq i+2)$ during time interval $[y_A, y_B]$ can occur if four following conditions are satisfied fully:

(1) the condition state i remains constant between a time point y_A to a time point $\bar{s}_i = y_A + \bar{z}_i \in [y_A, y_B]$.
(2) the condition state transits from i to $i+1$ at the time point $\bar{s}_i = y_A + \bar{z}_i$.
(3) the new condition state $i+1$ remains constant during the time interval $\bar{s}_i = y_A + \bar{z}_i$, $\bar{s}_{i+1} = \bar{s}_i + \bar{z}_{i+1}$ $(\leq y_B)$, and there is one new change from condition state from $i+1$ to $i+2$ at the end of this duration. Then keep following the same process.
(4) the condition state changes to j at the time point $\bar{s}_{j-1}$ $(\leq y_B)$ remains constant until the time point y_B.

If the entire process of transition is examined with the full satisfaction of all conditions, a conditional probability density function is given by:

$$q_j(\bar{z}_i, \bar{z}_{i+1}, \dots, \bar{z}_{j-1} | \xi_i \geq y_A) = g_i(\bar{z}_i | \xi_i \geq y_A) \prod_{m=i+1}^{j-1} f_m(\bar{z}_m) \tilde{F}_j \left(Z - \sum_{m=i}^{j-1} \bar{z}_m \right) \quad (2.21)$$

$$= \prod_{m=i}^{j-1} \theta_m . exp \left\{ - \sum_{m=i}^{j-1} \theta_m \bar{z}_m - \theta_j (Z - \sum_{m=i}^{j-1} \bar{z}_m) \right\}$$

$$= \prod_{m=i}^{j-1} \theta_m . exp \left\{ -\theta_j Z - \sum_{m=i}^{j-1} (\theta_m - \theta_j) \bar{z}_m \right\}$$

where $\bar{z}_i, \dots, \bar{z}_{j-1}$ are fixed values. Since the elapsed time or life expectancy ξ_i of condition state i $(i = 1, \dots, J-1)$ is a stochastic variable, the values of $z_i \geq 0, \dots, z_{j-1} \geq 0$ are variable with the following condition:

$$0 \leq z_i + z_{i+1} + \cdots + z_{j-1} \leq Z \quad (2.22)$$

Therefore, the Markov transition probabilities π_{ij} that a transition in the condition state from i to j $(j \geq i+2)$ occurs between two inspection or survey time points y_A and y_B or calendar time τ_{A} and τ_{B} can be specified. Tsuda et al (2006) had made great effort to make such sophisticated calculation for π_{ij}. Equation (2.23) shows the final result.

$$\pi_{ij} = Prob\,[h(y_B) = j | h(y_A) = i] \tag{2.23}$$

$$= \int_0^Z \int_0^{Z-z_i} \dots \int_0^{Z-\sum_{m=i}^{j-2} z_m} q_j\left(z_i, .., z_{j-1} \middle| \xi_i \geq y_A\right) dz_i \dots dz_{j-1}$$

$$= \sum_{k=i}^{j} \prod_{m=i}^{k-1} \frac{\theta_m}{\theta_m - \theta_k} \prod_{m=k}^{j-1} \frac{\theta_m}{\theta_{m+1} - \theta_k}\, exp(-\theta_k Z)$$

In summary, general forms of Markov transition probabilities based on exponential hazard model are given as following equations (Tsuda et al, 2006).

$$\pi_{ii} = exp(-\theta_i Z) \tag{2.24a}$$

$$\pi_{ii+1} = \frac{\theta_i}{\theta_i - \theta_{i+1}} \{-exp(-\theta_i Z) + exp(-\theta_{i+1} Z)\} \tag{2.24b}$$

$$\pi_{ij} = \sum_{k=i}^{j} \prod_{m=i}^{k-1} \frac{\theta_m}{\theta_m - \theta_k} \prod_{m=k}^{j-1} \frac{\theta_m}{\theta_{m+1} - \theta_k}\, exp(-\theta_k Z) \tag{2.24c}$$

$$\pi_{iJ} = 1 - \sum_{j=i}^{J-1} \pi_{ij} \tag{2.24d}$$

$$(i = 1, \dots, J-1)\ and\ (j \geq i+2).$$

2.3.4 Estimation of Markov Transition Probability

2.3.4.1 Contents of Periodical Inspection Data

Suppose periodical inspection data on the same category of K road pavement sections is available. An certain inspection sample k $(k = 1, \dots, K)$ consists of two periodical inspections or pavement condition surveys carried out at times τ_A^k and τ_B^k with the respective condition states ratings of $h(\tau_A^k)$ and $h(\tau_B^k)$. Differences in the inspection intervals of samples are

accepted. Based on the available set of inspection data, the inspection interval of a sample k is defined as $Z^k = \tau_B^k - \tau_A^k$. A dummy variable δ_{ij}^k $(i, j = 1, \dots, J; k = 1, \dots, K)$ that shows the deterioration progress patterns between two inspections times is defined as

$$\delta_{ij}^k = \begin{cases} 1 & when\ h(\tau_A^k) = i\ \ and\ \ h(\tau_B^k) = j \\ 0 & otherwise \end{cases} \tag{2.25}$$

Furthermore, the structural characteristics and usage conditions including climate factors that affect the deterioration of road pavement sections are represented by one vector ($\boldsymbol{x}^k = x_1^k, \dots, x_M^k$), where x_m^k $(m = 1, \dots, M)$ expresses the value of a characteristic variable m observed in the sample data k. The key information contained in the inspection sample data k can be rearranged as $\Xi^k = (\delta_{ij}^k, Z^k, \boldsymbol{x}^k)$. On the other hand, the exponential hazard function of the deterioration process for a sample data k $(k = 1, \dots, K)$ is

$$\lambda_i^k(y_i^k) = \theta_i^k \ \ (i = 1, \dots, J-1) \tag{2.26}$$

The hazard rate for condition state J is not defined because condition state J is absorption state and $\pi_{JJ} = 1$ as property of Markov chain. The hazard rate θ_i^k $(i = 1, \dots, J; k = 1, \dots, K)$ that expresses the deterioration process of road pavement section k can be changed due to the effects of relevant factors in the vector $\boldsymbol{x}^k$ as follow:

$$\theta_i^k = \boldsymbol{x}^k \boldsymbol{\beta}_i' \tag{2.27}$$

where $\boldsymbol{\beta}_i = (\boldsymbol{\beta}_{i,1}, \dots, \boldsymbol{\beta}_{i,M})$ is a row vector of unknown parameters $\boldsymbol{\beta}_{i,m}$ $(m = 1, \dots, M)$ and the symbol [′] indicates the transposed vector. To determine Markov transition probabilities, it is necessary to estimate the exponential hazard function $\lambda_i^k(y_i^k) = \theta_i^k$ based on the inspection sample information Ξ^k $(k = 1, \dots, K)$.

2.3.4.2 Life Expectancy of Condition State

By applying exponential hazard model, the expected elapsed period from the time the relevant condition state is reached until the following state can be determined as a result of the deterioration progress. Such period is understood as life expectancy or the remaining duration RMD_i^k of condition state i for the road pavement section k that can be defined by applying the survival function $\tilde{F}_i(y_i^k)$ in infinite domain (Lancaster, 1990)[4] as follow:

$$RMD_i^k = \int_0^\infty \tilde{F}_i\,(y_i^k) dy_i^k \tag{2.28}$$

By applying exponential hazard function to characterize the survival function $\tilde{F}_i(y_i^k)$ as shown in equation (2.12), the expected life expectancy of a condition state i for the road pavement section k becomes

$$RMD_i^k = \int_0^\infty exp(-\theta_i^k y_i^k)\, dy_i^k = \frac{1}{\theta_i^k} \tag{2.29}$$

The higher the hazard function, the shorter the life expectancy of condition state that leads to the reduction of pavement life.

The average life expectancy ET_j $(j = 2, \dots, J)$ of condition state j (> 1) is obtained by summation of life expectancy over condition state range counted from condition state $i = 1$ or the beginning of pavement cycle. Visual deterioration curve of infrastructure is thus drawn from obtained value of ET_j *(x)*.

$$ET_j = \sum_{i=1}^{j} \frac{1}{\theta_i^k} \tag{2.30}$$

Set of condition state i $(i = 1, \dots, J)$ and the average life expectancy ET_j $(j = 2, \dots, J)$ by the end of condition state j are utilized to make pavement deterioration curves in graph for illustration its deterioration process.

2.3.4.3 Estimation of the Hazard Model

Information contained in the inspection sample data $\Xi^k = (\bar{\delta}_{ij}^k, \bar{Z}^k, \bar{\boldsymbol{x}}^k)$ can be obtained in relation to the inspection sample k, where the symbol $[\,\bar{}\,]$ indicates an actual measured data. The Markov transition probabilities can be described in terms of the hazard functions as shown in equations (2.24a - 2.24d). Equation (2.27) also presents relationship between the hazard rate θ_i^k $(i = 1, \dots, J-1; k = 1, \dots, K)$ of each condition state that is contained in the Markov transition probabilities and the characteristic variables $\bar{\boldsymbol{x}}^k$ of pavement section. In addition, the transition probability also depends on inspection interval $\bar{Z}^k$.

The transition probability π_{ij} is expressed as a function of the measured data obtained from periodical inspections $(\bar{Z}^k, \bar{\boldsymbol{x}}^k)$ and the unknown parameters $\boldsymbol{\beta}_i$ as π_{ij} $(\bar{Z}^k, \bar{\boldsymbol{x}}^k : \boldsymbol{\beta}_i)$. If the deterioration progress of the pavement sections in a sample K are assumed to be mutually independent, the log-likelihood function expressing the simultaneous probability density of the deterioration transition pattern for entire inspection samples is [6], [7].

$$ln[\mathcal{L}(\boldsymbol{\beta})] = ln\left[\prod_{i=1}^{J-1}\prod_{j=i}^{J}\prod_{k=1}^{K}\left\{\pi_{ij}(\bar{Z}^k, \bar{\boldsymbol{x}}^k : \boldsymbol{\beta})\right\}^{\bar{\delta}_{ij}^k}\right] \tag{2.31}$$

$$= \sum_{i=1}^{J-1} \sum_{j=i}^{J} \sum_{k=1}^{K} \bar{\delta}_{ij}^{k} \, ln\left[\pi_{ij}(\bar{Z}^{k}, \bar{\boldsymbol{x}}^{k} : \boldsymbol{\beta})\right]$$

where $\bar{\delta}_{ij}^{k}, \bar{Z}^{k}$ and $\bar{\boldsymbol{x}}^{k}$ are all determined through inspections or surveys, and $\boldsymbol{\beta}_i (i = 1, \dots, J-1)$ are parameters to be estimated. Estimations of the parameters $\boldsymbol{\beta}$ can be obtained by solving the optimality conditions

$$\frac{\partial \, ln\left[\mathcal{L}(\widehat{\boldsymbol{\beta}})\right]}{\partial \, \boldsymbol{\beta}_{i,m}} = 0 \qquad (i = 1, \dots, J-1; m = 1, \dots, M) \tag{2.32}$$

that result from maximizing the log-likelihood function (2.31). The optimal values $\widehat{\boldsymbol{\beta}} = (\boldsymbol{\beta}_{1,1}, \dots, \boldsymbol{\beta}_{J,M})$ are then estimated by applying a numerical iterative procedure such as the Newton Method for the $(J-1)M$ order nonlinear simultaneous equations [8]. Furthermore, estimator for the asymptotic covariance matrix of the parameters is given by

$$\widehat{\boldsymbol{\Sigma}}(\widehat{\boldsymbol{\beta}}) = \left[\frac{\partial^2 \, ln\{\mathcal{L}(\widehat{\boldsymbol{\beta}})\}}{\partial \boldsymbol{\beta} \partial \boldsymbol{\beta}'}\right]^{-1} \tag{2.33}$$

The $(J-1)M \times (J-1)M$ order inverse matrix of the right-hand side of the above formula, composed by the elements $\partial^2 \ln\{\mathcal{L}(\widehat{\boldsymbol{\beta}})\} / \partial \boldsymbol{\beta}_{i,m} \partial \boldsymbol{\beta}_{i',m'}$ results to be the inverse matrix of the Fisher information matrix.

2.3.4.4 Time Adjustment and Average Markov Transition Probability

As shown in equations (2.24a - 2.24d), the Markov transition probabilities $(\pi_{ii}, \pi_{ii+1}, \pi_{ij}, \pi_{iJ})$ depend on the inspection interval Z. For convenience to follow and for emphasis, the Markov transition probability is expressed as $\pi_{ij}(Z)$, then the Markov transition probability matrix can be re-wrote taking into account of inspection interval Z as follow

$$\boldsymbol{\Pi}(Z) = \begin{pmatrix} \pi_{11}(Z) & \cdots & \pi_{1J}(Z) \\ \vdots & \ddots & \vdots \\ 0 & \cdots & \pi_{JJ}(Z) \end{pmatrix} \tag{2.34}$$

An integer number n is selected to examine Markov transition probabilities for two different cases in terms of inspection intervals Z and nZ. The Markov transition probability matrices $\boldsymbol{\Pi}(Z)$ and $\boldsymbol{\Pi}(nZ)$ explains the same deterioration process of certain sample for two different cases of interval of monitoring or periodical inspection. According to the law of matrix multiplication, the relation between $\boldsymbol{\Pi}(nZ)$ and $\boldsymbol{\Pi}(Z)$ is

$$\boldsymbol{\Pi}(nZ) = \{\boldsymbol{\Pi}(Z)\}^{n} \tag{2.35}$$

Given the vector $\boldsymbol{x}^k$ of the characteristic variables related to sample k, and the inspection interval Z^k, the Markov transition probabilities of a pavement section can be estimated by applying equations (2.24a - 2.24d). Markov transition probabilities satisfying time adjustment conditions can be estimated for arbitrary inspection intervals by changing the value Z^k.

In theory, Markov transition probabilities characterizing individual pavement section can be estimated at micro level. However in reality, when predicting the deterioration pattern of all road pavement sections as a whole, it is more practical to examine the average transition probabilities of the entire sample rather than transition probabilities for each individual section.

Under such situation, developing a methodology to estimate the average transition probabilities of the whole sample, which also satisfies the time adjustment conditions is needed. The hazard rate θ_i^k $(k = 1, \dots, K)$ should be utilized taking into account of the distribution of characteristic variable $\boldsymbol{x}$. The distribution function for the population sample of pavement sections is denoted as $\Gamma(\boldsymbol{x})$.

Then, the expected value of hazard rate $E[\theta_i]$ subject to the entire population sample can be defined as:

$$E[\theta_i] = \int_{\Theta} \boldsymbol{x}\boldsymbol{\beta}_i' d\,\Gamma(\boldsymbol{x}) \tag{2.36}$$

where Θ makes reference to the entire population sample. A Markov transition probability matrix is realized to satisfy the time adjustment conditions if 1) employing exponential hazard equations (2.24a - 2.24d) are used to estimate Markov transition probabilities, and 2) the Markov transition probability matrix for each sample is defined by the hazard rate $\theta_i^k (i = 1, \dots, J-1; k = 1, \dots, K)$. Accordingly, the Markov transition probability matrix estimated by applying equation (2.36) also satisfies the time adjustment conditions.

2.4 LOCAL MIXTURE MARKOV DETERIORATION HAZARD MODEL

2.4.1 Introduction

Further to the application of Markov chain model in PMS, other prominent studies have addressed the application of some statistic distribution such as Weibull, Gamma, Poison, and Exponential distribution families [2], [9], [10]. In addition, in an effort to capture the hazard rates of different groups in an infrastructure system, heterogeneity factor is embedded in the Markov chain model. The Markov hazard model with embedded heterogeneity factor is named as "Mixture hazard model" [11], [12].

In the practice of stochastic estimation, the heterogeneity factor is assumed to follow a stochastic distribution. Attempt to apply Gamma distribution for heterogeneity factor in

probabilistic approach has been implemented in a larger scale information system, where the breakdown of informative devices follows Weibull function [13]. This probabilistic approach can also be used to apply on the pavement system. However, the assumption of heterogeneity factor in probabilistic manner may not always reflect the reality. And as the matter of course, an explicit form for the numerical computation may be difficult to acquire. This problem can be regarded as a limitation of probabilistic approach.

In order to overcome the limitation of probabilistic approach, local mixture model is recommended as an alternative solution [11], [14]. In the local mixture hazard model, the distribution of heterogeneity factor is to follow Taylor series, which eventually reduces the computational complexity. However, to date, it has not been seen in the literature of reliability engineering and operations research with a full scale study on the mixture hazard model, especially in combination with the Markov chain model.

A recent study on risk management in pavement system using local mixture model has attracted attention for benchmarking study on pavement management system [15]. This study employed Markov chain model for estimating the hazard rate and Markov transition probability. However, it remains with several limitation such as a full scale of benchmarking study was not targeted. Further, empirical study focused only on a small numbers of samples. Thus, the aim of this paper is to propose a comprehensive estimation methodology of local mixture model and its applicability for benchmarking purposes.

2.4.2 Markov Transition Probability and Heterogeneity Factor

In reality, it is hard to grant a homogeneous sampling population in monitoring data. To express the inhomogeneous sampling population, a great deal of research in reliability engineering and operations research employs the term "heterogeneity factor". In PMS, it is assumed that the entire network of roads in analysis consists of K groups in total of road sections. The grouping classification is often based on the differences in technology. In each group k $(k = 1, \dots, K)$, there are S_k road sections in total. The heterogeneity factor of an individual group is denoted as ε^k, which infers the change of hazard rate of condition state i $(i = 1, \dots, J-1)$ with respect to the pavement section s_k $(s_k = 1, \dots, S_k)$. With this assumption, the formula of hazard function in equation (2.8) can be expressed by means of the mixture form

$$\lambda_i^{S_k} = \tilde{\lambda}_i^{S_k} \varepsilon^k \quad (i = 1, \dots, J-1; k = 1, \dots, K; s_k = 1, \dots, S_k) \tag{2.37}$$

It is noted that the value of heterogeneity factor ε^k is always non-negative. In addition, if the value of heterogeneity factor ε^k for group k is higher than that of other groups, the group k certainly has a faster deterioration than other groups. Within one group, the hazard rate of all condition states share a same value of heterogeneity factor ε^k. Within a network of roads

categorized by several groups, the distribution of heterogeneity factor ε^k reflects the influence of individual group on the overall deterioration of the entire network. Depending of the natural characteristics of each road network system, the heterogeneity factor can be defined as in the form of a function or a stochastic variable.

For measurable representation, the value of $\varepsilon^k (k = 1, \dots, K)$ is described by vector $\bar{\varepsilon}^k$, with the bar $\bar{}$ indicating the measurable value. As a result, the survival probability function in equation (2.12) can be further expressed by means of the mixture hazard rate in equation (2.37):

$$\tilde{F}_i(y_i^k) = exp(-\tilde{\lambda}_i \bar{\varepsilon}^k y_i^k) \tag{2.38}$$

Similarly, the Markov transition probability expressed in equations (2.17) and (2.23) can be further defined as

$$\pi_{ii}^k(z^k : \bar{\varepsilon}^k) = exp(-\tilde{\lambda}_i^k \bar{\varepsilon}^k z^k) \tag{2.39}$$

$$\pi_{ii}^k(z^k : \bar{\varepsilon}^k) = \sum_{l=i}^{j} \prod_{m=i,\neq l}^{j-1} \frac{\tilde{\lambda}_m^k}{\tilde{\lambda}_m^k - \tilde{\lambda}_l^k} \, exp(-\tilde{\lambda}_l^k \varepsilon^k z^k) \tag{2.40}$$

$$= \sum_{l=i}^{j} \psi_{ij}^l(\tilde{\boldsymbol{\lambda}}^k) \, exp(-\tilde{\lambda}_l^k \varepsilon^k z^k)$$

$$(i = 1, \dots, I-1; j = i+1, \dots, I; k = 1, \dots, K),$$

where

$$\psi_{ij}^l(\tilde{\lambda}^k) = \prod_{m=i,\neq l}^{j-1} \frac{\tilde{\lambda}_m^k}{\tilde{\lambda}_m^k - \tilde{\lambda}_l^k} \tag{2.41}$$

A great deal of past research has revealed the difficulties in defining the heterogeneity factor ε^k. The assumption of the heterogeneity factor to be in the form of a function or a stochastic variable crucially depends on the characteristics of the system itself and the availability of monitoring data [11], [14]. This research [15] focuses on applying mixture model in the case that the value distribution of heterogeneity factor ε^k has a small dispersion. In other words, the departure of heterogeneity factor ε^k from homogeneity is in a small scale. This type of mixture model is named as the local mixture model. In exponential family form $f(x; \varepsilon)$ (where x and ε are the variable and heterogeneity respectively), local mixing mechanism is defined via its mean parameterization μ^k

$$g(x;\mu) := f(x;\varepsilon) + \sum_{i=2}^{r} f^k(x;\varepsilon) \tag{2.42}$$

where

$$f^k(x;\varepsilon) = \frac{\delta^k}{\delta\varepsilon^k} f(x;\varepsilon)$$

Expansion of functions in equations (2.42) can be recognized to follow the Taylor series. Since the Likelihood function of Markov transition probability in (2.42) belongs to the exponential family. It is possible to approximate the transition probability as in the form of the local mixture distribution.

$$\tilde{\pi}_{ij}(z) = \int_0^\infty \pi_{ij}\,(z{:}\,\varepsilon) f(\varepsilon) d\varepsilon \quad (i = 1, \dots, J-1) \tag{2.43}$$

For convenience of mathematical manipulation, the local mixture transition probability is assumed as an exponential function $f_{mix}(\varepsilon, z, \lambda)$, with mix indicating the abbreviation of mixture. As the sequent, the mixture function $f_{mix}(\varepsilon, z, \lambda)$ can be described by means of standard function $f(\varepsilon, z, \lambda)$ and distribution $H(\varepsilon)$. Equation (2.43) is further simplified as

$$f_{mix}(\varepsilon, z, \lambda) = \int f(\varepsilon, z, \lambda)\, dH(\varepsilon) \tag{2.44}$$

where $f(\varepsilon, z, \lambda) = \exp\,(-\varepsilon\lambda z)$. Function $f(\varepsilon, z, \lambda)$ is likely a function of ε about its mean. Without no loss of generality, and as long as the mean exist, we can further decompose equation (2.42) as follows:

$$exp(-\varepsilon\lambda z) = e^{-\lambda z}(1 + (\varepsilon - 1)(-\lambda z) + \frac{(\varepsilon - 1)^2}{2!}(-\lambda z)^2 + \cdots \tag{2.45}$$

This is the Taylor series. And thus, the quadratic form (when $r = 2$) is acceptable for an accurate approximation. Consequently, an explicit form of approximation can be derived for the Markov transition probability:

$$E\left(e^{-\varepsilon\lambda z}\right) \approx e^{-\lambda z}\left\{1 + \frac{(\sigma\lambda z)^2}{2}\right\} \tag{2.46}$$

and

$$\tilde{\pi}_{ii}(z) = e^{-\tilde{\lambda}_i z}\left\{1 + \frac{\left(\sigma\tilde{\lambda}_i z\right)^2}{2!}\right\} \tag{2.47}$$

$$\tilde{\pi}_{ij}(z) = \sum_{l=i}^{j} \psi_{ij}^{l}\left(\tilde{\lambda}\right) e^{-\tilde{\lambda}_l z}\left\{1 + \frac{\left(\sigma\tilde{\lambda}_l z\right)^2}{2!}\right\} \tag{2.48}$$

$$(i = 1, \dots, J-1; j = i+1, \dots, J)$$

2.4.3 Likelihood Estimation Approach

2.4.3.1 Estimation Assumption

The estimation of Markov transition probability and heterogeneity factor require monitoring data from at least two visual inspections. Supposing that the periodical monitoring data of S_k road sections is available. An inspection sample s_k (a road section) implies two consecutive discrete periodical inspections at times $\bar{\tau}_A^{s_k}$ and $\bar{\tau}_B^{s_k} = \bar{\tau}_A^{s_k} + \bar{z}^{s_k}$, with its respective condition states $h\left(\bar{\tau}_A^{s_k}\right) = i$ and $h\left(\bar{\tau}_B^{s_k}\right) = j$. Based on monitoring data of $\sum_{k=1}^{K} S_k$ samples, dummy variable $\bar{\delta}_{ij}^{s_k}(i = 1, \dots, J-1, j = i, \dots, J; s_k = 1, \dots, S_K; k = 1, \dots, K)$ is defined to satisfy the following conditions

$$\bar{\delta}_{ij}^{s_k} = \begin{cases} 1 & h\left(\bar{\tau}_A^{s_k}\right) = i\,, \quad h\left(\bar{\tau}_B^{s_k}\right) = j \\ 0 & otherwise \end{cases} \tag{2.49}$$

The range of dummy variable $(\bar{\delta}_{11}^{s_k}, \dots, \bar{\delta}_{J-1,J}^{s_k})$ is denoted by using the dummy variable vector $\bar{\boldsymbol{\delta}}^{s_k}$. Furthermore, structural characteristics and environment conditions of the road are expressed by means of characteristic variable vector $\bar{\boldsymbol{x}}^{s_k} = (\bar{x}_1^{s_k}, \dots, \bar{x}_M^{s_k})$ with $\bar{x}_m^{s_k}(m = 1, \dots, M)$ indicating the observed value of variable m for sample s_k. The first variable is referred as a constant term, with its value $x_1^{s_k} = 1$. Thus, the information concerning monitoring data of sample k can be described as $\boldsymbol{\Xi}^{s_k} = \left(\bar{\boldsymbol{\delta}}^{s_k}, \bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}\right)$.

The hazard rate of condition state i of sample s_k can be expressed by using mixture hazard function $\lambda_i^{s_k}\left(y_i^{s_k}\right) = \tilde{\lambda}_i^{s_k}\varepsilon^k(i = 1, \dots, J-1$, with J as the absorbing condition state satisfying the conditions $\pi_{JJ}^{s_k} = 1$ and $\tilde{\lambda}_J^{s_k} = 0$. The hazard rate $\tilde{\lambda}_i^{s_k}(i = 1, \dots, J-1; s_k = 1, \dots, L_k)$ depends on the characteristic vector of the road section, and is described as follows

$$\tilde{\lambda}_i^{s_k} = \boldsymbol{x}^{s_k}\boldsymbol{\beta}_i' \tag{2.50}$$

where $\boldsymbol{\beta}_i = (\beta_{i,1}, \dots, \beta_{i,M})$ is a row vector of unknown parameters $\beta_{i,m} = (m = 1, \dots, M)$, and the symbol [′] indicates the transposed vector. From equations (4.47) and (4.48), the

standard hazard rate of respective condition states can be expressed by means of hazard rate $\tilde{\lambda}_i^{s_k}(i = 1, \dots, J-1; s_k = 1, \dots, L_k)$ and heterogeneity parameter ε^k. The average Markov transition probability can be expressed in equation (2.48), with consideration of characteristic variable $\bar{\boldsymbol{x}}^{s_k}$. In addition, the transition probability depends on inspection interval $\bar{z}^{s_k}$. As a result, transition probability π_{ij} can be expressed as a function of measurable monitoring data $(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k})$ and unknown parameter $\boldsymbol{\theta} = (\boldsymbol{\beta}_1, \dots, \boldsymbol{\beta}_{J-1}, \sigma)$ as $\tilde{\pi}_{ij}^{s_k}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}: \boldsymbol{\theta})$. If the deterioration of road sections l_k in the entire L_K samples are assumed to be mutually independent, the Likelihood function expressing the simultaneous probability density of the deterioration transition pattern for all inspection samples is defined [17], [18]:

$$\boldsymbol{\mathcal{L}}(\boldsymbol{\theta}, \boldsymbol{\Xi}) = \prod_{i=1}^{J-1} \prod_{j=i}^{J} \prod_{k=1}^{K} \prod_{s_k=1}^{S_k} \{\tilde{\pi}_{ij}^{s_k}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}: \boldsymbol{\theta})\}^{\bar{\delta}_{ij}^{s_k}} \tag{2.51}$$

In view of the local mixture distribution with Taylor series, the Markov transition probability can be further described as

$$\tilde{\pi}_{ii}^{s_k}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}: \boldsymbol{\theta}) = e^{-\bar{\boldsymbol{x}}^{s_k}\boldsymbol{\beta}_i'\bar{z}^{s_k}} \left\{1 + \frac{(\sigma^{s_k}\bar{\boldsymbol{x}}^{s_k}\boldsymbol{\beta}_i'\bar{z}^{s_k})^2}{2!}\right\} \tag{2.52}$$

$$\tilde{\pi}_{ij}^{s_k}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}: \boldsymbol{\theta}) = \sum_{l=i}^{j} \psi_{ij}^{l}(\tilde{\boldsymbol{\lambda}}) e^{-\bar{\boldsymbol{x}}^{s_k}\boldsymbol{\beta}_l'\bar{z}^{s_k}} \left\{1 + \frac{(\sigma^{s_k}\bar{\boldsymbol{x}}^{s_k}\boldsymbol{\beta}_l'\bar{z}^{s_k})^2}{2!}\right\} \tag{2.53}$$

$$(i = 1, \dots, J-1; j = i+1, \dots, J).$$

where $\psi_{ij}^{s}(\tilde{\boldsymbol{\lambda}}^{l_k})$ is referred to equation (2.41). Since $\tilde{\delta}_{ij}^{s_k}, \bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}$ are known from inspection, then $\hat{\boldsymbol{\theta}} = (\hat{\boldsymbol{\beta}}, \hat{\sigma})$ can be estimated by applying maximum likelihood approach. Likelihood function in equation (2.51) can be re-wrote by means of logarithm as follows

$$ln\, \boldsymbol{\mathcal{L}}(\boldsymbol{\theta}, \boldsymbol{\Xi}) = \sum_{i=1}^{J-1} \sum_{j=1}^{J} \sum_{k=1}^{K} \sum_{s_k=1}^{S_k} \tilde{\delta}_{ij}^{s_k} \tilde{\pi}_{ij}^{s_k}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k}: \boldsymbol{\theta}) \tag{2.54}$$

The estimation of $\boldsymbol{\theta}$ can be obtained by solving the optimality conditions

$$\frac{\partial ln\, \boldsymbol{\mathcal{L}}(\boldsymbol{\theta}, \boldsymbol{\Xi})}{\partial \theta_i} = 0, \quad (i = 1, \dots, (J-1)M + 1) \tag{2.55}$$

The, the optimal value of $\hat{\boldsymbol{\theta}} = (\hat{\theta}_{\mathbf{1}}, \dots, \hat{\theta}_{(J-1)M+1})$ are estimated by applying a numerical iterative procedure such as Newton Method for the $(J-1)M + 1$ order nonlinear

simultaneous equations [8]. Furthermore, the estimator for the asymptotical covariance matrix $\widehat{\boldsymbol{\Sigma}}(\widehat{\boldsymbol{\theta}})$ of the parameters is given by

$$\widehat{\boldsymbol{\Sigma}}(\widehat{\boldsymbol{\theta}}) = \left[\frac{\partial^2 ln\, \boldsymbol{\mathcal{L}}(\widehat{\boldsymbol{\theta}}, \boldsymbol{\Xi})}{\partial \boldsymbol{\theta} \partial \boldsymbol{\theta}'}\right]^{-1} \tag{2.56}$$

The $\{(J-1)M+1\} \times \{(J-1)M+1\}$ order inverse matrix of the right-hand side of the formula, composed by the elements $\partial^2 \boldsymbol{\mathcal{L}}(\boldsymbol{\theta}, \Xi)/\boldsymbol{\partial}\theta_i \boldsymbol{\partial}\theta_j$ results to be the inverse matrix of the Fisher information matrix.

2.4.3.2 Heterogeneity Factor Estimation

Information concerning inspection sample s_k of pavement group k is denoted as $\boldsymbol{\xi}^{s_k}(s_k = 1, \dots, S^k)$. In order to describe the condition states of individual sample, the first and second condition states of sample s_k are assumed as $i(s_k)$ and $j(s_k)$. It is supposed that the parameter set $\widehat{\boldsymbol{\theta}} = (\widehat{\boldsymbol{\beta}}_1, \dots, \widehat{\boldsymbol{\beta}}_{J-1}, \hat{\sigma})$ is available. If the distribution of heterogeneity factor ε^k in function $\bar{f}(\varepsilon : \hat{\sigma})$ is considered, the probability density function, which infers the transition pattern of sample $\boldsymbol{\xi}^{s_k}$ can be defined as

$$\rho^{s_k}(\varepsilon^k : \widehat{\boldsymbol{\theta}}, \boldsymbol{\xi}^k) = \left\{\pi^{s_k}_{i(s_k)j(s_k)}(\bar{z}^{s_k}, \bar{\boldsymbol{x}}^{s_k} : \widehat{\boldsymbol{\beta}}, \varepsilon^k)\right\}^{\bar{\delta}^{s_k}_{i(s_k)j(s_k)}} \bar{f}(\varepsilon^k, \hat{\sigma}) \tag{2.57}$$

where function $\bar{f}(\varepsilon^k : \hat{\sigma})$ follows local mixing mechanism as previously described. As for the total number of samples in group k, the probability density function concerning the simultaneous occurrence of transition can be further defined as

$$\rho^k(\varepsilon^k : \widehat{\boldsymbol{\theta}}, \boldsymbol{\xi}^k) = \prod_{s_k=1}^{S^k} \rho^k(\varepsilon^k : \widehat{\boldsymbol{\theta}}, \boldsymbol{\xi}^k) \propto \tag{2.58}$$

$$\prod_{s_k=1}^{S_k} \left\{\sum_{l=i(s_k)}^{j(s_k)} \psi^l_{i(s_k)j(s_k)}\left(\tilde{\boldsymbol{\lambda}}^{s_k}(\widehat{\boldsymbol{\theta}})\right) \exp\left(-\tilde{\lambda}^{s_k}_l(\widehat{\boldsymbol{\theta}})\varepsilon^k\right)\bar{z}^{s_k})\right\}^{\bar{\delta}^{s_k}_{i(s_k)j(s_k)}} \left\{1 + \frac{\left(\sigma\tilde{\lambda}^{s_k}_l z^{s_k}\right)^2}{2!}\right\}^{s_k}$$

The standard or average hazard rate is expressible by means of vector $\tilde{\boldsymbol{\lambda}}^{s_k}(\widehat{\boldsymbol{\theta}}) = \left(\tilde{\lambda}^{s_k}_1(\widehat{\boldsymbol{\theta}}), \dots, \tilde{\lambda}^{s_k}_{J-1}(\widehat{\boldsymbol{\theta}})\right)$. With this assumption, the value of average hazard rate $\tilde{\lambda}^{s_k}_1$ depends on the value of parameter $\widehat{\boldsymbol{\theta}}$. To come up with an explicit form of the probability density function in equation (2.58), we apply partial logarithm as follows:

$$\ln \rho^k\left(\varepsilon^k : \widehat{\boldsymbol{\theta}}, \xi^k\right) \propto \sum_{s_k=1}^{S^k} \bar{\delta}_{i(s_k)j(s_k)}^{s_k}$$
$$\ln\left\{\sum_{l=i(s_k)}^{j(s_k)} \psi_{i(s_k)j(s_k)}^{l}\left(\tilde{\boldsymbol{\lambda}}^{s_k}\left(\widehat{\boldsymbol{\theta}}\right)\right)\exp\left(-\tilde{\lambda}_l^{s_k}\left(\widehat{\boldsymbol{\theta}}\right)\varepsilon^k\right)\bar{z}^{s_k}\right)\right\} \tag{2.59}$$
$$+ S_k ln\left\{1 + \frac{\left(\sigma\tilde{\lambda}_l^{s_k} z^{s_k}\right)^2}{2!}\right\}$$

The last, optimal value of heterogeneity factor $\varepsilon^k (k = 1, \dots, K)$ can be obtained through maximizing equation (2.59) with respect to ε^k as variable and $\widehat{\boldsymbol{\theta}} = (\widehat{\boldsymbol{\beta}}_1, \dots, \widehat{\boldsymbol{\beta}}_{l-1}, \widehat{\boldsymbol{\sigma}})$:

$$max_{\varepsilon^k}\left\{ln\ \rho^k\left(\varepsilon^k : \widehat{\boldsymbol{\theta}}, \xi^k\right)\right\} \tag{2.60}$$

2.4.4 Benchmarking Flowchart and Utilization

The objective of benchmarking study is to examine different deterioration processes of various groups of samples. This research focuses on road pavement. There are variety pavement types have been applied properly on different categories of classes of roads. Best performance pavements should be paved on high class roads of expressways or trunk roads. On the other hand, in case of low class roads in rural area or farming with minimum requirement on its performance, low cost pavement is usually the best seclection to simplify the burden on budget. For same pavement type, there can be some available technologies that make it different. On one route, same pavement structure can be laid on various kind of foundations or subgrades due to geological conditions, ground water operation, changing climate conditions along the road and so forth that leads to the different of its performance also. In one rather long route or a road network, it is nealy impossible to see the homogeneous operation conditions in terms of traffic, loading, maintenance, etc., also.

All such about situations lead to the differences in deterioration process of road pavements. By setting some criteria for homogeneous sections, it is feasible to prepare groups of homogeneous sections that can be evaluated its deterioration speeds by applying local mixture markov deterioration hazard model to estimate heterogeneity factors.

Based on the methodology proposed in the previous sections, flowchart of of benchmarking application for pavement deterioration evaluation is proposed as shown in Figure 2.5.

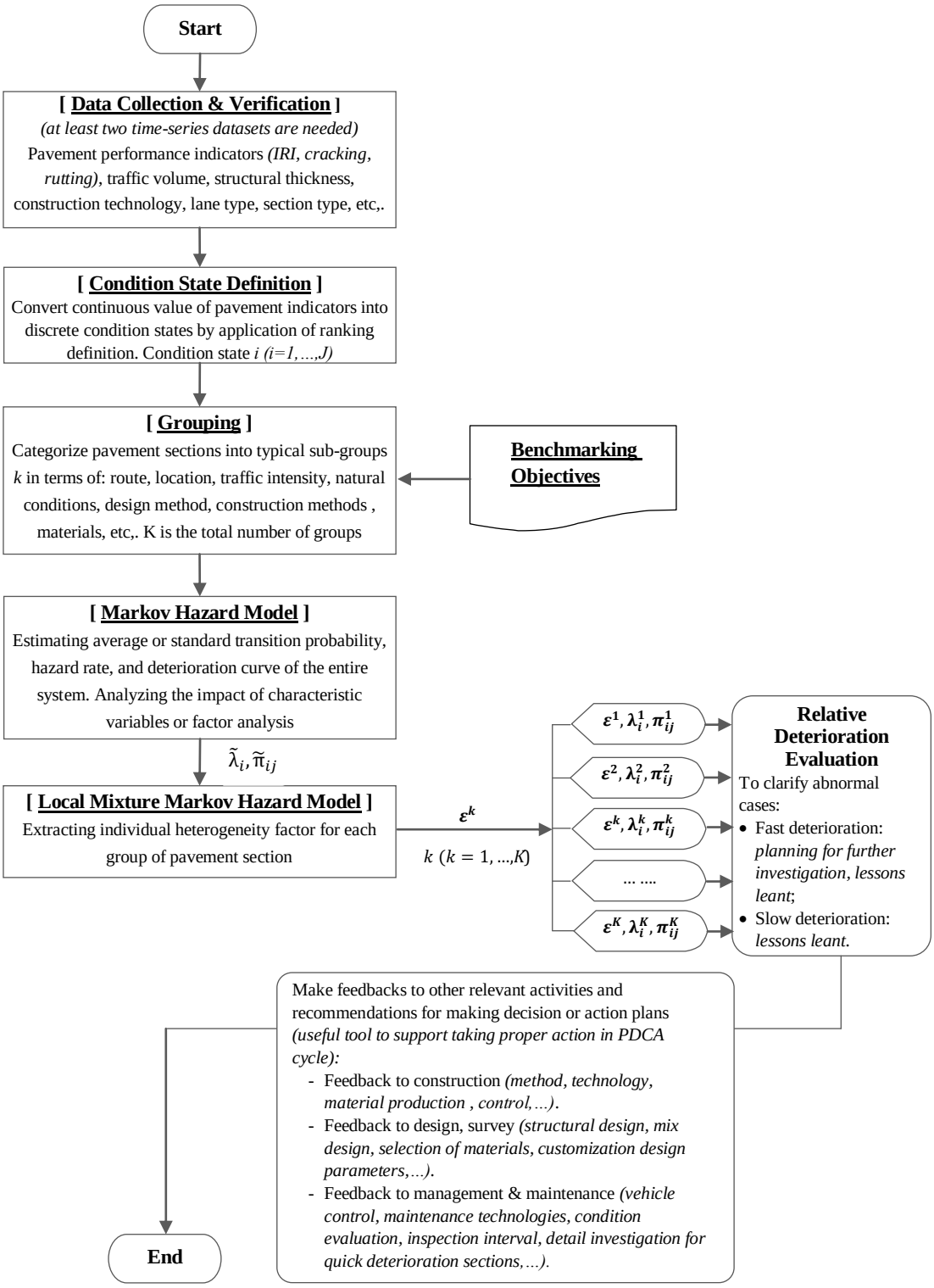


Figure 2.5 Benchmarking Flowchart by the Local Mixture Hazard Model

For the entire samples, using Markov hazard model, average or standard transition probability in $\tilde{\pi}_{ij}$ matrix, average hazard rate $\tilde{\lambda}_i$ can be estimated that are used to calculate life expectancy RMD_i and specify deterioration curve of the entire system or the benchmark case or the case $\varepsilon = 1$ in Figure 2.6.

Benchmarking evaluation to compare deterioration speeds of pavement groups is made by applying local mixture hazard model. For each group k of pavement sections, information vector $(\varepsilon^k, \lambda_i^k, \pi_{ij}^k)$ is estimated. The bigger the heterogeneity factor ε^k of an individual group, the faster deterioration speed occurring in pavement sections belong to the group.

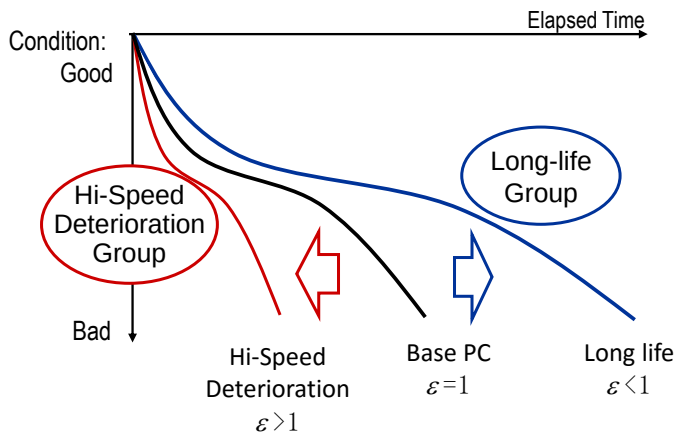


Figure 2.6 Benchmarking of Pavement Deterioration Speed

There is no doubt about great effects from benchmarking analysis because it can quantitatively point out the critical groups of sections with fast or abnormal deterioration that need being investigated further to specify the causes and proper treatments. For pavements, intensive and structural surveys of loading capacity like FWD test should be performed just for these candidate sections instead of doing tests for all sections including many sections in good condition. The principle of classification of data items into different hierarchical levels for utilization and collection has been realized by applying benchmarking analysis that supports for optimization of data collection in both terms of cost and time (Figure 2.7).

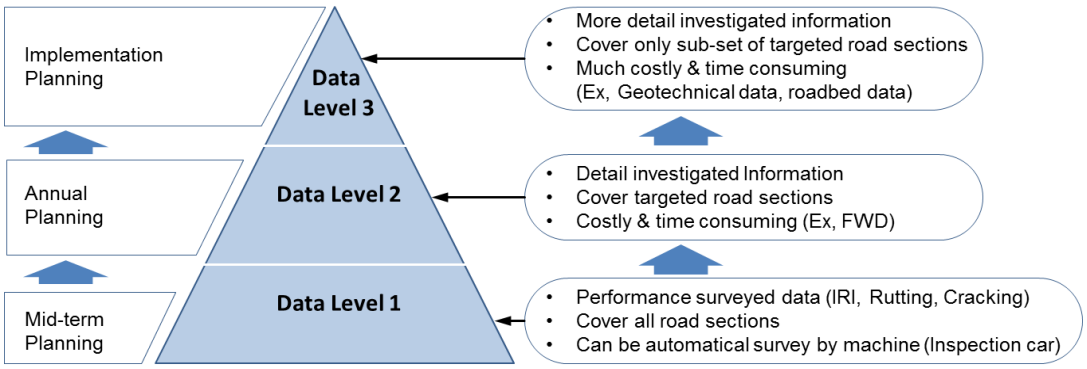


Figure 2.7 Classification of Pavement Data Items into Hierarchical Levels

Depending on benchmarking objectives, there are also various utilization of benchmarking evaluation results. For administrator who is responsible for management of a road network, one of his biggest concern is the overall deterioration image of the entire network to make decisions at macro level. All information obtaining, synthesizing from data, analysis results

relevant to road management must be so significant that can be used to make right feedbacks for enhancement of any related activities of surveying, designing, construction and maintenance also. In the PDCA cycle, benchmarking analysis belongs to the third part "Check" of the cycle that support for taking strategic actions. Continuously keep repeating implementation the cycle is the principle of enhancing all practices in road sector.

2.5 EMPIRICAL STUDY OF BENCHMARKING IN PAVEMENT APPLICATION

2.5.1 Overview

As the description in Chapter 5, there are many national roads in the road network that cover the whole one-dimentional country of Vietnam. And in the road network, there are many inhomogeneous factors of natural conditions such as: traffic, terrain, geological conditions, hydraulic conditions, temperature, rainfall, ground water operation, and so forth that may cause the differences in deterioration of asphalt concrete pavement in the routes.

The objective of this case study is to examine pavement deterioration speeds in different national routes that is expected to provide useful information and quantitative evaluation to road administrators for making proper decision and taking right action plans. Thus, the exponential hazard model for benchmarking evaluation to estimate the Markov transition probability is applied. Further, the heterogeneity factor of individual route is estimated by using the local mixture model. Benchmarking study is highlighted with the comparison of deterioration curves.

Empirical application is conducted on the monitoring data of the national road system in Vietnam. During the period of HDM-4 application in Vietnam, pavement surveys had been conducted to formulate datasets for system operation. There were also huge samples of data in the database surveyed that were mostly collected in 2001, 2004 and 2007. However, due to the poor consistence and low reliability of these time-series data, it has not been selected for utilization in this study.

Within framework of JICA Project on Capacity Enhancement in Road Maintenance in Vietnam, pavement condition survey applying automatic technology by inspection car and data processing had been conducted from February 2012 and March 2013 to collect data for national routes in the northern Vietnam with total length of 2,303 km corresponding to 4,606 lane-kilometers in both directions. Data is recorded and stored continuously during the survey in the field to be processed automatically later on in the office except pavement crack ratio to formulate pavement condition dataset for all single sections of 100 lane-meters long.

The dataset had been formulated based upon the latest pavement condition survey in 2012 with the coverage of 21 national highways in the northern Vietnam. Preparation of another

time-series dataset as the requirement of Markov hazard model to estimate transition probabilities had been conducted based on maintenance history data with the assumption to set the best condition state for pavement at the completion time of new construction or the latest big repair (Figure 2.8).

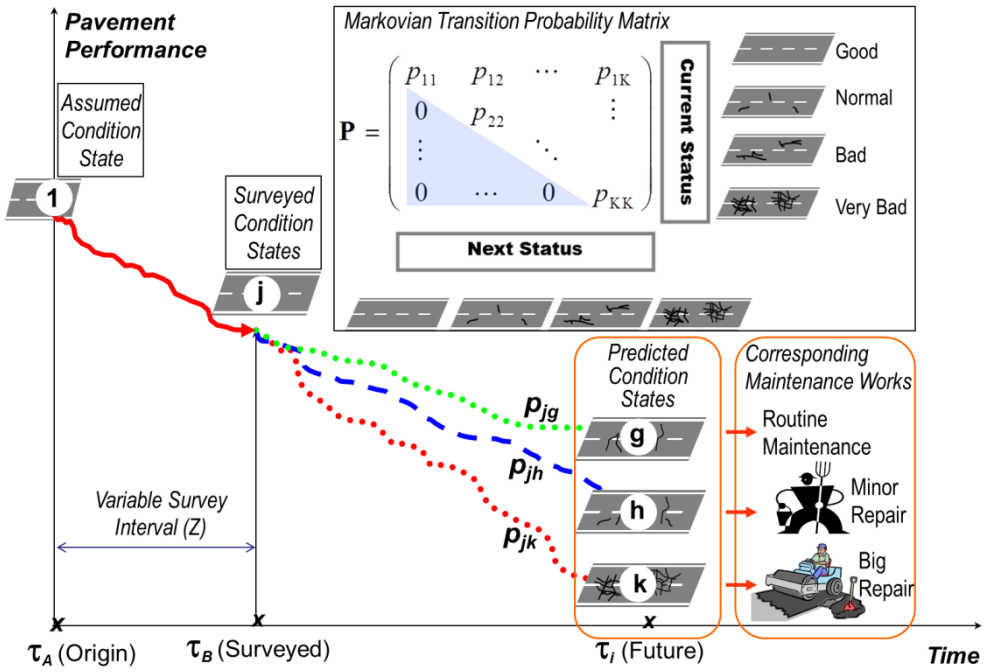


Figure 2.8 Illustration of Markov transition probability theory

Beside three pavement indicators of IRI, rutting depth, cracking ratio, some other data are also collected during survey such as road coordinators and road front images with capture interval of five meters, locations of road facilities, and some inventory data. The overall pavement index of MCI (Chapter 5) is also calculated from the three individual indicators to evaluate pavement soundness and the need of repair works.

Table 2.1 Definition of ranks or condition states for pavement conditions

Rank	Range of pavement indicator or index values				Remark
	Rutting (mm)	IRI (m/km)	Crack ratio (%)	MCI	
1	[0; 10)	[0; 2)	0	(8; 10]	Very good
2	[10; 20)	[2; 4)	[0; 10)	(6; 8]	Good
3	[20; 30)	[4; 6)	[10; 20)	(4; 6]	Fair
4	[30; 40)	[6; 8)	[20; 30)	(2; 4]	Poor
5	[40; 50)	≥ 8	[30; 40)	≤ 2	Very poor
6	≥ 50		≥ 40		Serious

2.5.2 Estimation results and Benchmarking Analysis

Definition of ranks or condition states for pavement conditions representing by three individual indicators and overall index was shown on Table 2.1 to apply Markov hazard model and local mixture hazard model as well.

Pavement deterioration evaluation module analyzes pavement deterioration progress or the transition of pavement condition states or pre-defined rankings. Markov transition probability theory is applied in the analysis to calculate the transition probabilities from a certain ranking to other rankings of pavement deterioration based on periodically observed pavement conditions to specify pavement deterioration rate and its life expectancy in each ranking and in total.

In the benchmarking study, the estimation results for heterogeneity factor of individual group are given in Table 2.2.

Table 2.2 Distribution of Heterogeneity Factors in terms of Road Names

No.	Road Name	Number of sample	Heterogeneity factor (Epsilon ε) for distress			
			Rutting	Roughness	Crack	Combined distress (MCI)
0	BM	47820	1	1	1	1
1	NH.1	5627	0.57	0.42	0.36	0.56
2	RR3	62	1.16	1.31	0.13	0.97
3	NH.2	6090	1.13	0.91	1.19	1.23
4	NH.3	6298	1.19	1.47	0.98	0.82
5	NH.4E	481	0.67	0.80	0.98	1.15
6	NH.5	1825	1.22	0.61	0.64	1.20
7	NH.6	7451	1.02	0.96	1.09	1.02
8	NH.6.2	84	0.91	1.71	0.89	1.04
9	NH.6.3	291	1.25	2.85	2.55	2.10
10	NH.10	2948	0.84	0.88	0.19	0.66
11	NH.1.1	130	7.57	3.78	0.47	5.99
12	NH.15	410	0.78	1.25	0.78	0.82
13	NH.18	1042	0.34	0.50	0.44	0.49
14	NB-BN	784	0.44	0.57	0.13	0.53
15	HCM	2150	1.19	1.11	1.20	1.43
16	NH.37	767	0.40	0.84	0.93	0.62
17	NH.38	1514	0.76	1.44	1.48	1.00
18	NH.38B	1656	1.89	3.88	3.80	1.76
19	NH.43	1141	0.93	1.17	0.98	1.09
20	NH.70	4351	2.12	1.75	1.71	1.60
21	NH.279	2718	1.39	1.49	1.82	1.76

In comparison with the benchmark or average of the entire samples case with the heterogeneity factor epsilon ε=1 for distress, some roads have smaller hazard rates that show the good

performance of pavement with longer life. Conversely, there can be seen the fast deterioration in some roads. The most critical case in pavement deformation of rutting and roughness is found in the branch of highway No.1 (NH.1.1) that connects Ninh Phuc Port to national highway No.1 with the total length of thirteen kilometers.

Estimated results of hazard rate and life expectancy for each condition state for individual group of road pavement in term of road name are shown in Table 2.3, Table 2.4, Table 2.5, Table 2.6 as the outcomes of application local mixture hazard model.

Pavement deterioration curves in terms of individual distress of rutting, roughness, and cracking and in combined distress characterizing by MCI were created that visually show the transition of pavement condition states from the best condition to the worst one through its service life (Figure 2.9, Figure 2.10, Figure 2.11, Figure 2.12).

With the definition of pavement physical life at the rutting depth of 50mm as the starting point of condition state $i = 6$, we can determine average life (15.53 years) of pavements for the whole samples of sections in 21 national roads in the northern area as the benchmark case shown in Table 2.3, Figure 2.9 . National road NH.18 is the best performance among all with its life (45.23 years) is nearly three times longer than the benchmark case. It is followed by NH.37 and Noi Bai - Bac Ninh routes with the life of 38.37 years and 35.08 years, respectively. More than half of routes have the life spans distributing around the average value. Deterioration of pavements in NH.38B and NH.70 is very fast in comparison with the benchmark case with the life reduction in half. However, the absolutely abnormal case in pavement deterioration is found in branch route NH.1.1. It seems unbelievable for pavement engineers, and be so hard to accept such result. Just taking a look at the illustrative graph in Figure 2.9, it is quite sure that all road administrators will raise the same question of why such situation had occurred. At this state and based on the results of benchmarking analysis, important decisions will be made. One of the decisions is on intensive data collection or in-depth investigation to identify the causes of all abnormal cases with fast deterioration and the proper treatment for a certain number of specified candidate sections instead of for the whole samples.

In case of roughness, as shown in Table 2.1, pavement physical life is defined with the critical condition state $i = 5$ at the value of IRI is equal or higher than 8m/km. The average pavement life of the whole samples of sections in 21 national roads in the northern area is 18.49 years as shown in Table 2.4 and Figure 2.10. The best performance is national road NH.1 with its life in roughness is 43.75 years. NH.5, NH.18, and Noi Bai-Bac Ninh route are three best cases among remaining routes with the life longer than 30 years that is nearly double the benchmark case in case of NH.18. While the life of other remaining are distributed around average value, three cases of NH.6.3 (one branch on NH.6), NH.1.1 (one branch of NH.1), and NH.38B show the worst of pavement performance with the loss of pavement life is around two third in comparison with the average case of the whole sample.

Cracking is one typical pavement distress that has different failure mechanism in comparison with pavement deformation in rutting and roughness. With the definition of pavement physical life at the cracking ratio of 40 percent of pavement surface as the starting point of condition state $i = 6$, we can determine average life (18.93 years) of pavements for the whole samples of sections in 21 national roads in the northern area as the benchmark case shown in Table 2.5 and Figure 2.11. National road NH.1 is the best case of pavement performance among all with its life (52.62 years) is nearly three times longer than the benchmark case. It is followed by NH.1.1 and NH.18 with the life of 40.41 years and 42.8 years, respectively. The most critical cases of fast deterioration can be saw in NH.6.3 and NH.38B with the bad pavement condition with alligator cracking on the surface at the time of survey in 2012 and currently also.

Table 2.3 Estimated Life expectancy of condition states and pavement life in RUTTING

No.	Road Name	Hazard rate θ_i in RUTTING for condition state					Survival time in RUTTING in condition state (years)					Pavement life (years)
		1	2	3	4	5	1	2	3	4	5	
-	BM	1.08	0.2	0.2	0.37	0.58	0.93	5.07	5.13	2.69	1.72	**15.53**
1	NH.1	0.62	0.11	0.11	0.21	0.33	1.62	8.85	8.95	4.7	3	**27.13**
2	RR3	1.26	0.23	0.23	0.43	0.68	0.8	4.35	4.4	2.31	1.48	**13.34**
3	NH.2	1.22	0.22	0.22	0.42	0.66	0.82	4.47	4.53	2.38	1.52	**13.71**
4	NH.3	1.28	0.23	0.23	0.44	0.69	0.78	4.26	4.32	2.27	1.45	**13.08**
5	NH.4E	0.72	0.13	0.13	0.25	0.39	1.39	7.58	7.67	4.03	2.57	**23.23**
6	NH.5	1.32	0.24	0.24	0.45	0.71	0.76	4.15	4.2	2.21	1.41	**12.72**
7	NH.6	1.1	0.2	0.2	0.38	0.59	0.91	4.95	5.01	2.63	1.68	**15.18**
8	NH.6.2	0.98	0.18	0.18	0.34	0.53	1.02	5.57	5.64	2.96	1.89	**17.08**
9	NH.6.3	1.35	0.25	0.24	0.46	0.73	0.74	4.06	4.11	2.16	1.38	**12.46**
10	NH.10	0.91	0.17	0.16	0.31	0.49	1.1	6.03	6.1	3.21	2.05	**18.49**
11	NH.1.1	8.17	1.49	1.48	2.81	4.4	0.12	0.67	0.68	0.36	0.23	**2.05**
12	NH.15	0.85	0.15	0.15	0.29	0.46	1.18	6.46	6.54	3.44	2.19	**19.82**
13	NH.18	0.37	0.07	0.07	0.13	0.2	2.7	14.75	14.93	7.84	5.01	**45.23**
14	NB-BN	0.48	0.09	0.09	0.16	0.26	2.09	11.44	11.58	6.08	3.88	**35.08**
15	HCM	1.28	0.23	0.23	0.44	0.69	0.78	4.26	4.32	2.27	1.45	**13.08**
16	NH.37	0.44	0.08	0.08	0.15	0.24	2.29	12.51	12.67	6.65	4.25	**38.37**
17	NH.38	0.82	0.15	0.15	0.28	0.44	1.22	6.65	6.73	3.54	2.26	**20.39**
18	NH.38B	2.04	0.37	0.37	0.7	1.1	0.49	2.68	2.71	1.42	0.91	**8.21**
19	NH.43	1	0.18	0.18	0.34	0.54	1	5.45	5.52	2.9	1.85	**16.72**
20	NH.70	2.28	0.42	0.41	0.79	1.23	0.44	2.39	2.42	1.27	0.81	**7.34**
21	NH.279	1.5	0.27	0.27	0.52	0.81	0.67	3.64	3.68	1.94	1.24	**11.16**

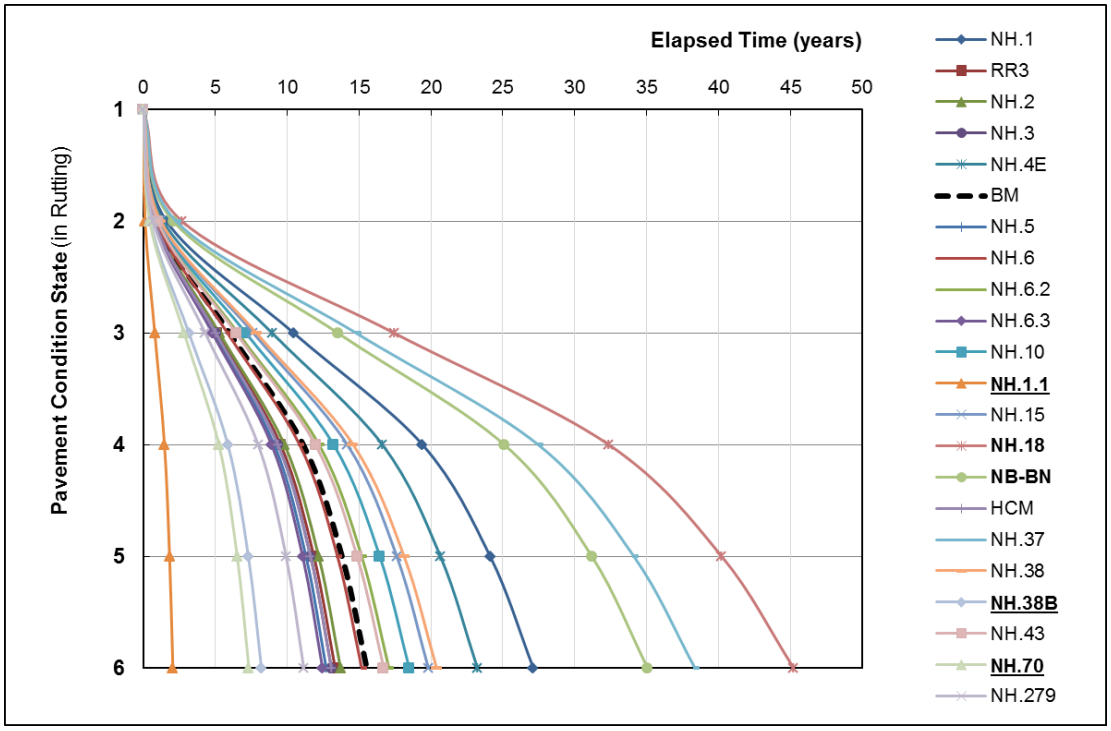


Figure 2.9 Estimated Pavement Performance Curves in RUTTING of Different Roads

Table 2.4 Estimated Life expectancy of condition states and pavement life in ROUGHNESS

No.	Road Name	Hazard rate θ_i in ROUGHNESS for condition state				Survival time in ROUGHNESS in condition state (years)				Pavement life (years)
		1	**2**	**3**	**4**	**1**	**2**	**3**	**4**	
-	BM	0.5	0.1	0.26	0.47	1.99	10.49	3.88	2.13	**18.49**
1	NH.1	0.21	0.04	0.11	0.2	4.71	24.82	9.18	5.04	**43.75**
2	RR3	0.66	0.12	0.34	0.61	1.52	8.03	2.97	1.63	**14.15**
3	NH.2	0.46	0.09	0.23	0.43	2.19	11.54	4.27	2.34	**20.34**
4	NH.3	0.74	0.14	0.38	0.69	1.36	7.14	2.64	1.45	**12.59**
5	NH.4E	0.4	0.08	0.21	0.38	2.48	13.08	4.84	2.66	**23.05**
6	NH.5	0.31	0.06	0.16	0.29	3.25	17.12	6.33	3.47	**30.17**
7	NH.6	0.48	0.09	0.25	0.45	2.06	10.88	4.02	2.21	**19.17**
8	NH.6.2	0.86	0.16	0.44	0.8	1.17	6.14	2.27	1.25	**10.83**
9	NH.6.3	1.43	0.27	0.73	1.34	0.7	3.68	1.36	0.75	**6.49**
10	NH.10	0.44	0.08	0.23	0.41	2.25	11.88	4.39	2.41	**20.94**
11	NH.1.1	1.9	0.36	0.98	1.78	0.53	2.77	1.03	0.56	**4.89**
12	NH.15	0.63	0.12	0.32	0.59	1.59	8.38	3.1	1.7	**14.77**
13	NH.18	0.25	0.05	0.13	0.24	3.96	20.86	7.71	4.24	**36.77**
14	NB-BN	0.28	0.05	0.15	0.27	3.52	18.54	6.85	3.76	**32.67**
15	HCM	0.56	0.11	0.29	0.52	1.79	9.42	3.48	1.91	**16.6**
16	NH.37	0.42	0.08	0.22	0.4	2.36	12.46	4.61	2.53	**21.96**
17	NH.38	0.72	0.14	0.37	0.67	1.39	7.3	2.7	1.48	**12.87**
18	NH.38B	1.95	0.37	1	1.82	0.51	2.7	1	0.55	**4.76**
19	NH.43	0.59	0.11	0.3	0.55	1.71	8.99	3.32	1.82	**15.84**
20	NH.70	0.88	0.17	0.45	0.82	1.14	5.99	2.22	1.22	**10.56**
21	NH.279	0.75	0.14	0.38	0.7	1.33	7.03	2.6	1.43	**12.39**

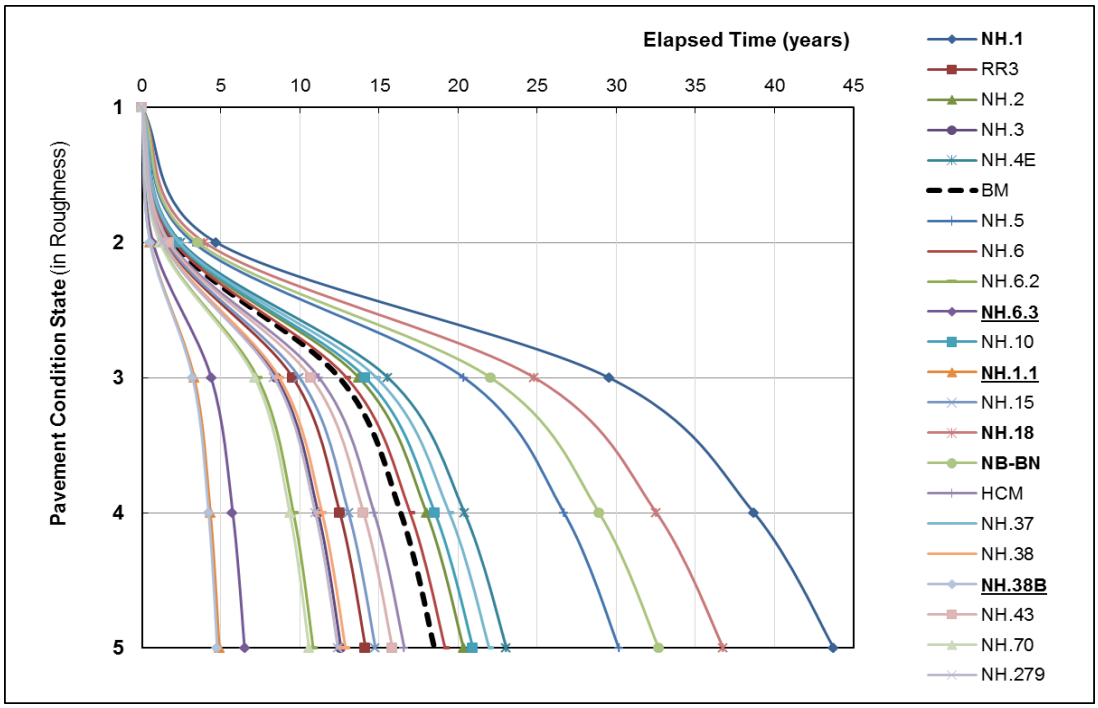


Figure 2.10 Estimated Pavement Performance Curves in ROUGHNESS of Different Roads

Table 2.5 Expected Life expectancy of condition states and pavement life in CRACKING

No.	Road Name	Hazard rate θ_i in CRACKING for condition state					Survival time in CRACKING in condition state (years)					Pavement life (years)
		1	2	3	4	5	1	2	3	4	5	
1	BM	0.09	0.21	0.65	1.18	1.58	11.16	4.75	1.54	0.85	0.63	**18.93**
2	NH.1	0.03	0.08	0.23	0.43	0.57	31.01	13.21	4.29	2.35	1.75	**52.62**
3	NH.2	0.11	0.25	0.77	1.4	1.88	9.42	4.01	1.3	0.71	0.53	**15.98**
4	NH.3	0.09	0.21	0.63	1.15	1.55	11.43	4.87	1.58	0.87	0.65	**19.4**
5	NH.4E	0.09	0.21	0.64	1.16	1.56	11.37	4.84	1.57	0.86	0.64	**19.29**
6	NH.5	0.06	0.13	0.41	0.75	1.01	17.48	7.44	2.42	1.33	0.99	**29.65**
7	NH.6	0.1	0.23	0.71	1.29	1.73	10.23	4.36	1.41	0.78	0.58	**17.36**
8	NH.6.2	0.08	0.19	0.57	1.05	1.4	12.61	5.37	1.74	0.96	0.71	**21.39**
9	NH.6.3	0.23	0.54	1.65	3.01	4.03	4.38	1.87	0.61	0.33	0.25	**7.44**
10	NH.1.1	0.04	0.1	0.3	0.55	0.74	23.82	10.14	3.29	1.81	1.35	**40.41**
11	NH.15	0.07	0.16	0.51	0.92	1.24	14.31	6.1	1.98	1.09	0.81	**24.29**
12	NH.18	0.04	0.09	0.29	0.52	0.7	25.23	10.74	3.49	1.91	1.43	**42.8**
13	HCM	0.11	0.25	0.78	1.42	1.9	9.29	3.96	1.28	0.7	0.53	**15.76**
14	NH.37	0.08	0.19	0.6	1.09	1.47	12.04	5.13	1.66	0.91	0.68	**20.43**
15	NH.38	0.13	0.31	0.96	1.75	2.35	7.53	3.21	1.04	0.57	0.43	**12.78**
16	NH.38B	0.34	0.8	2.46	4.49	6.02	2.94	1.25	0.41	0.22	0.17	**4.98**
17	NH.43	0.09	0.21	0.64	1.16	1.55	11.38	4.85	1.57	0.86	0.64	**19.3**
18	NH.70	0.15	0.36	1.11	2.02	2.71	6.53	2.78	0.9	0.5	0.37	**11.08**
19	NH.279	0.16	0.38	1.18	2.15	2.89	6.12	2.61	0.85	0.46	0.35	**10.38**

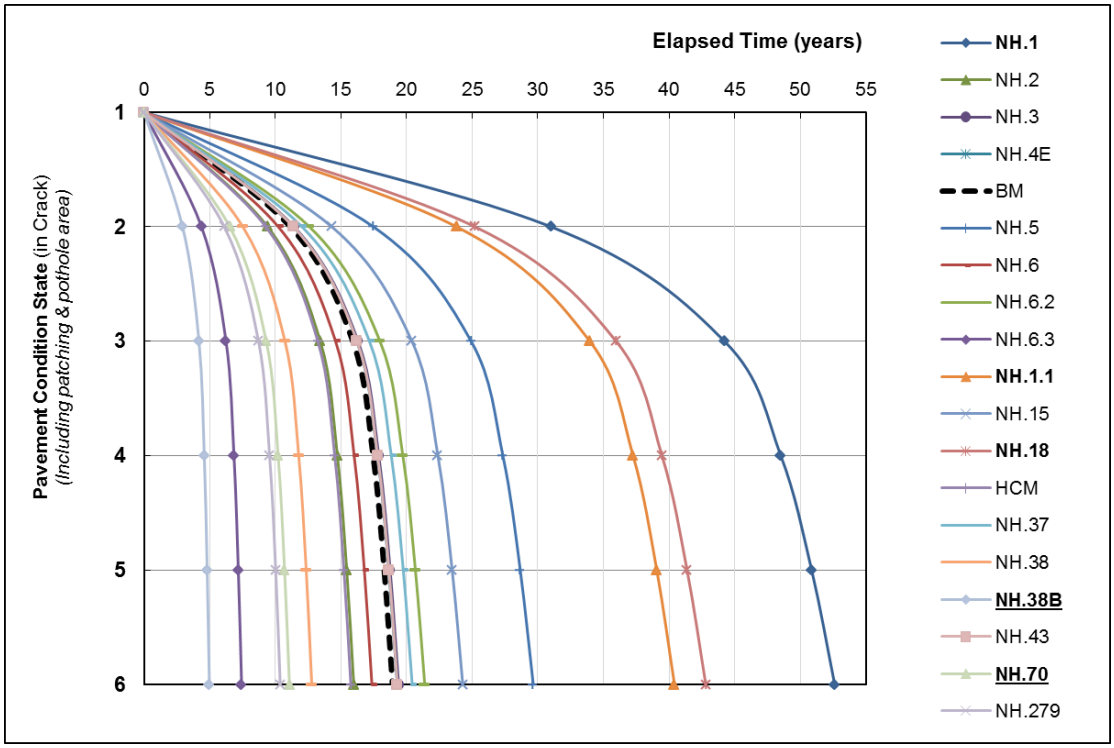


Figure 2.11 Estimated Pavement Performance Curves in CRACKING of Different Roads

As shown in Table 2.3, Table 2.4, Table 2.5 and Figure 2.9, Figure 2.10, Figure 2.11, pavement deterioration in individual distress has been estimated. There exist differences of deterioration process or life expectancy more or less among national roads and among types of pavement distresses also.

There are various types of distresses or defects in asphalt concrete pavement. However, the occurrence of these distresses is not simultaneous sometime due to the typical characteristics of asphalt concrete pavement and its mechanism of failures. Therefore, beside the study on prediction of pavement deterioration in individual distress, this research also challenges to study on pavement index (Chapter 5) and application of local mixture hazard model to evaluate pavement performance taking into account of combined distresses instead of individual failure.

MCI has been selected to be the pavement index. Based on the definition of pavement condition or performance in term of MCI, there are five condition states $(i = 1, \dots, 5)$ that characterizes pavement performance from the best condition $(i = 1)$ to the worst case $(i = 5)$. Results of estimation are shown in Table 2.6 and Figure 2.12.

Table 2.6 Expected Life expectancy of condition states and pavement life in MCI

No.	Road Name	Hazard rate θ_i in combined index MCI for condition state				Survival time in combined index MCI in condition state (years)				Pavement life (years)
		1	**2**	**3**	**4**	**1**	**2**	**3**	**4**	
-	BM	0.58	0.1	0.18	0.15	1.73	9.58	5.64	6.68	**23.63**
1	NH.1	0.32	0.06	0.1	0.08	3.11	17.2	10.13	12	**42.44**
2	RR3	0.56	0.1	0.17	0.15	1.79	9.88	5.82	6.89	**24.37**
3	NH.2	0.71	0.13	0.22	0.18	1.41	7.78	4.58	5.42	**19.19**
4	NH.3	0.47	0.09	0.15	0.12	2.11	11.65	6.86	8.13	**28.75**
5	NH.4E	0.66	0.12	0.2	0.17	1.5	8.31	4.89	5.8	**20.51**
6	NH.5	0.69	0.13	0.21	0.18	1.44	7.97	4.69	5.56	**19.66**
7	NH.6	0.59	0.11	0.18	0.15	1.7	9.41	5.54	6.56	**23.21**
8	NH.6.2	0.6	0.11	0.18	0.16	1.67	9.24	5.44	6.44	**22.79**
9	NH.6.3	1.21	0.22	0.37	0.31	0.83	4.56	2.69	3.18	**11.26**
10	NH.10	0.38	0.07	0.12	0.1	2.63	14.54	8.56	10.14	**35.86**
11	NH.1.1	3.45	0.62	1.06	0.9	0.29	1.6	0.94	1.12	**3.95**
12	NH.15	0.47	0.09	0.15	0.12	2.12	11.7	6.89	8.16	**28.87**
13	NH.18	0.28	0.05	0.09	0.07	3.55	19.61	11.55	13.68	**48.39**
14	NB-BN	0.31	0.06	0.09	0.08	3.28	18.11	10.66	12.63	**44.67**
15	HCM	0.82	0.15	0.25	0.21	1.22	6.71	3.95	4.68	**16.56**
16	NH.37	0.36	0.07	0.11	0.09	2.78	15.38	9.06	10.73	**37.95**
17	NH.38	0.58	0.1	0.18	0.15	1.73	9.58	5.64	6.68	**23.64**
18	NH.38B	1.01	0.18	0.31	0.26	0.99	5.44	3.21	3.8	**13.43**
19	NH.43	0.63	0.11	0.19	0.16	1.59	8.8	5.18	6.14	**21.72**
20	NH.70	0.92	0.17	0.28	0.24	1.08	5.98	3.52	4.17	**14.76**
21	NH.279	1.02	0.18	0.31	0.26	0.98	5.43	3.2	3.79	**13.41**

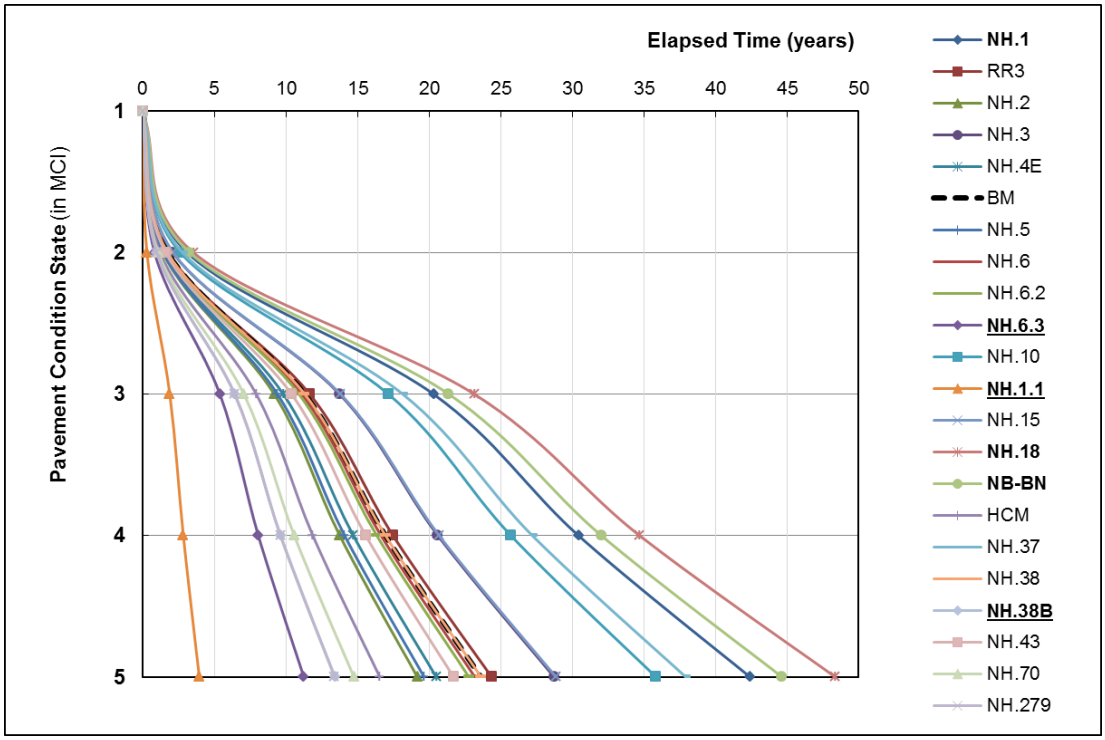


Figure 2.12 Estimated Pavement Performance Curves in MCI of Different Roads

Based on all estimated results, it is necessary to make comparative table that shows pavement life forecasting in different types of its distresses as follow.

Table 2.7 Estimated Life expectancy in terms of pavement distresses

No.	Road Name	Number of Samples	Estimated Life Expectancy in			
			Rutting	Roughness	Cracking	MCI
0	BM	47820	15.53	18.49	18.93	23.63
1	NH.1	5627	27.13	43.75	52.62	42.44
2	NH.2	6090	13.71	20.34	15.98	19.19
3	NH.3	6298	13.08	12.59	19.40	28.75
4	NH.4E	481	23.23	23.05	19.29	20.51
5	NH.5	1825	12.72	30.17	29.65	19.66
6	NH.6	7451	15.18	19.17	17.36	23.21
7	NH.6.2	84	17.08	10.83	21.39	22.79
8	NH.6.3	291	12.46	6.49	7.44	11.26
9	NH.1.1	130	2.05	4.89	40.41	3.95
10	NH.15	410	19.82	14.77	24.29	28.87
11	NH.18	1042	45.23	36.77	42.80	48.39
12	HCM	2150	13.08	16.60	15.76	16.56
13	NH.37	767	38.37	21.96	20.43	37.95
14	NH.38	1514	20.39	12.87	12.78	23.64
15	NH.38B	1656	8.21	4.76	4.98	13.43
16	NH.43	1141	16.72	15.84	19.30	21.72
17	NH.70	4351	7.34	10.56	11.08	14.76
18	NH.279	2718	11.16	12.39	10.38	13.41

By integrating all individual results into one table, it is more rational to make proper evaluation of pavement condition. It is no doubt about the good pavement performance with slow deterioration in NH.18 with the high values of its individual life expectancies.

NH.1 shows very good performance in cracking resistance with the physical life of 52.62 years. However, it seems not proper to make conclusion of perfect pavement performance of the road due to its shorter life in rutting distress with the reduction nearly in half. Moreover, these results have been estimated based on just data of pavement condition on the light traffic lanes. Field verification shows the rutting situation in reality on the heavy traffic lanes in more serious situation.

Among these three indicators, rutting and roughness reflect stability of asphalt concrete with very high sensitiveness to high temperature. The more unstable asphalt concrete, the less elastic modulus or rigidity that reduces material defection and deformation resistance and also increases bending capacity that prevents pavement from cracking. In the worst case, bleeding can be seen on pavement surface, pavement seriously deteriorates with high rutting depth and critical roughness but no sign of cracking. Therefore, long performance in cracking resistance of NH.1 in Figure 2.11, Table 2.5 under the fact of destructive deformation is meaningless to make evaluation of pavement quality or its life.

The case of NH.1.1 is much more meaning to show the necessary to seriously take all pavement distresses into account for proper and sufficient evaluation its performance. Taking a look at the Table 2.7, it is quite clear that pavement on the route is totally damaged in serious deformation in both transversal and longitudinal directions. Such typical type of defect on asphalt pavement represents poor stability of asphalt concrete under loading and other factors. Even in some area, bleeding can be found. It becomes quite natural that crack seems unobservable in this case. Therefore, long life expectancy in cracking (40.41 years) has no contribution to pavement quality in this case.

Pavement deterioration in NH.6.3 and NH.38B shows one different case: rutting is less serious than cracking and roughness. In this case, it can be confirmed that the typical distress on the pavement is cracking at critical level that make serious reduction its structural integration and becomes very potential for penetration of drain water into the foundation or subgrade. Deformation in large scale should be the common consequence that is the cause for increasing of pavement roughness. Rutting or transversal vertical deformation is not so sensitive with the occurrence of big deformed area on the surface.

2.6 SUMMARY AND RECOMMENDATIONS

An in-depth literature review on hazard model practice in infrastructure asset management in general and pavement asset management in particular has been described in this chapter. The

methodology to estimate Markov transition probabilities to forecast the deterioration of road pavements was also discussed. The transition progress among a set of condition states representing the conditions of each pavement section were defined by using exponential hazard models with its parameters are estimated by applying the maximum likelihood method.

Besides, this chapter has also explained a local mixture model for benchmarking study. The local mixture model is expressed by means of heterogeneity factor ε that exists in each group of pavements. The heterogeneity factor is considered to follow the function of Taylor series. In order to estimate the factor, two steps estimation approach with maximum likelihood estimation method is applied. The local mixture hazard model is considered as an excellent tool for benchmarking study, which is used to support for making the optimum decisions in road management and maintenance in various types such as: search for the best technologies in pavement design, materials, constructions and maintenance through post-evaluation; finding the most suitable application conditions for pavement technologies that can be traffic intensity, vehicle's axle load, geotechnical conditions, hydraulic and ground water conditions and so forth.

Empirical study applying local mixture hazard model has been researched to identify the differences of pavement deterioration process in different national roads. The estimated results of exponential hazard models, pavement life expectancies in all major pavement indicators and its overall index gives one comprehensive image for road administrators to make proper evaluation about road conditions, its soundness or quality to take right actions.

Further study on improvement practices of pavement engineering in Vietnam with the focus on enhancement of pavement design specifications by applying hazard models and local mixture hazard model is presented in Chapter 6.

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CHAPTER 3

Compound Hidden Markov Deterioration Models for Pavement Structure

3.1 INTRODUCTION

The soundness level of a road surface directly affects the service level for road users. When the soundness level of a road surface is deteriorated, maintenance and repair of the road surface, such as overlay, are conducted to recover pavement serviceability. On the other hand, the load bearing capacity of the overall pavement structure (surface, base foundation, subgrade, and roadbed) is also deteriorated by repeated loads and other natural factors. Decrease in the load bearing capacity also increases the deterioration speed of the road surface performance. Consequently, not only maintenance and repair of the road surface, but also repair of the overall pavement structure (hereafter, pavement renewal) is necessary. In order to manage the pavement, it is necessary to forecast its deterioration based on the simultaneous deterioration of the surface and pavement strength or its load bearing capacity.

Deterioration of the pavement is a combination of phenomena with different deterioration mechanisms: road surface deterioration and decrease in the pavement load bearing capacity. The decrease in the load bearing capacity of the pavement may cause defects to the surface like roughness, rutting, cracking or increase deterioration speed. Therefore, for a section with severely deteriorated pavement structure, deterioration speed of the surface also accumulatively increases. From this perspective, Kobayashi et al.[1] proposed the hierarchical hidden Markov deterioration model, in which the deterioration speed of the road surface depends on the deterioration condition of the pavement load bearing capacity. However, this one-sided relation does not completely represent the relation between the pavement surface soundness level and its load bearing capacity. The progress of deterioration of the road surface may deteriorate the load bearing capacity also. This study formulates a compound deterioration process, considering mutual interaction between road surface deterioration and decrease in load bearing capacity of the pavement, as a compound Markov deterioration model.

Regarding the deterioration of road surface, its soundness level can be measured with pavement condition survey using contactless technologies like laser or camera. For the case of pavement load bearing capacity, information on structural deterioration conditions should be determined by investigation testing such as FWD test. These two surveys are commonly

independent and there are few cases of simultaneous implementation due to the requirement for optimal survey or investigation in hierarchical levels as shown in Figure 2.7 in Chapter 2. Therefore, it is difficult to simultaneously obtain data on the soundness level and load bearing capacity of road pavement. To create the compound Markov deterioration model, it is necessary to formulate a compound hidden Markov deterioration model, which demonstratively considers the mechanism of data missing from the results of the pavement condition and FWD surveys, and develops a method to estimate these data.

Being aware of such problem, this study formulates a compound hidden Markov deterioration model, which demonstratively considers systematic absent data. Additionally, the study also proposes a method that allow the estimation of model parameters with the MCMC (Markov chain Monte Carlo) method based on the results of pavement condition and FWD surveys, which are not conducted simultaneously. Section 3.2 describes the fundamental of this study. Section 3.3 formulates the compound Markov deterioration model, and Section 3.4 proposes an estimation method based on the compound hidden Markov deterioration model. MCMC based algorithm is described in Section 3.5. Finally, Section 3.6 demonstrates and analyzes the algorithm with missing data for one empirical study. Effectiveness of the model proposed in this study is also discussed in the section.

3.2 BASIC OF THE STUDY

3.2.1 Outline of Conventional Research

The Markov deterioration model has been proposed as a statistical deterioration forecast model for social infrastructure facilities. One method to estimate a Markov transition probability is the counting of actual data on the transition condition between soundness levels. Additionally, Sugisaki et al.[2] proposed a method to collectively estimate the Markov transition probability through visual inspection of data from different inspection intervals. Next, the introduction of the hazard analysis technique [3],[4] drastically developed the estimation of the Markov transition probability. Mishalani and Madanat [5] proposed a method to express the Markov transition probability with the exponential hazard model, targeting only two adjacent soundness levels. Independently, Tsuda et al.[6] proposed a multi-stage exponential hazard model that expressed the condition of transition between two or more optional soundness levels, in addition to a general method to estimate the Markov transition probability. After that, the multi-stage Weibull hazard model [7] was proposed to estimate the inhomogeneous Markov transition probability containing the past record, and the competitive Markov deterioration model [8],[9] was proposed that expressed the process of transition between the different deterioration patterns. Concerning the estimation method of the Markov transition probability, the Bayesian deterioration models [10],[11] which allow estimation of the Markov

transition probability by coupling an engineer's experimental information with measurement results for a phase in which only a few measurement samples were available. The following methods were proposed: the method to correct a missing bias generated by missing measurement data with preventive repair [12]; the random proportional Weibull hazard model [13], which considers the heterogeneity of the hazard rate; and the mixed Markov deterioration hazard model [14]. Additionally, Kobayashi et al.[15] proposed the hidden Markov deterioration model to estimate the Markov transition probability, targeting a case in which the soundness level included a measurement error. Concerning the deteriorating process of the pavement in this study, Kobayashi et al.[1] simulated two types of Markov processes, including the deterioration process of the road surface and the decrease in the load bearing capacity of the pavement, and expressed a mechanism in which the decrease in the load bearing capacity affected the deterioration process of the pavement surface performance with the hierarchic hidden Markov deterioration model. However, the reverse mechanism in which the deterioration progress of the pavement surface performance takes negative influence to the load bearing capacity had not been considered. In this study, we expand the hierarchic hidden Markov deterioration hazard model [1] and develop the compound Markov deterioration hazard model, based on the consideration of mutual interaction between the surface deterioration progress and the demonstrative decrease in the load bearing capacity of the road pavement. The measurably influences to each other have been clarified in the study. Simultaneous unavailability of the soundness level and load bearing capacity data of the road surface was took into account with the assumption of observation environment in which only one of these two types of data can be acquired at each survey time point. Then, we formulate the compound hidden Markov deterioration hazard model, which is systematically based on the assumption that part of the inspection data is missing. There is no existing research that proposes this composite Markov deterioration hazard model and estimation method. The compound hidden Markov deterioration model proposed in this study represents a method to estimate the deterioration model through simulating the multiple types of deterioration processes and different inspection results. It is expected to expand its application for more social infrastructure facilities other than pavements in the future.

3.2.2 Compound Deterioration Process

The pavement of a road is a multi-layered structure, including surface, base and sub-base layers, subgrade and roadbed. In this study, the deterioration condition of a pavement is expressed through two evaluation indices: (i) the soundness level of the road surface and (ii) the load bearing capacity of the overall pavement. The road surface deteriorates continuously due to abrasion caused by vehicles, repeated loads, and direct influences of weather and temperature. In addition, if there is structural defect in layers, the deterioration speed of pavement surface is affected, which may cause surface cracks, wheel track rutting, or deterioration in the form of roughness or deflection as well. Through the road condition survey,

pavement individual indicators or overall index can be determined to indicate the soundness level of its surface. On the other hand, the base layer and subgrade also deteriorate due to vehicle accumulative loads especially under critical conditions of surface water penetration or high water content due to rainfall, ground water infiltration. When a crack or damaged part of the road surface becomes more serious, the possibility of deep penetration into the pavement increases that leads to the structural dis-integrity. Therefore, the progress of surface deterioration leads to more rapid deterioration of the foundation and subgrade. Unlike regular pavement condition surveys, the load bearing capacity of the pavement cannot be observed visually. In order to determine the decrease in pavement structure loading capacity, it is necessary to conduct destructive test, such as core sampling or measure pavement deflection under loading in non-destructive tests such as FWD or Benkelman beam deflection tests. In the FWD survey, pavement load bearing capacity can be diagnosed by dropping a mass on the pavement surface to measure the generated deflections that show pavement deflection basin.

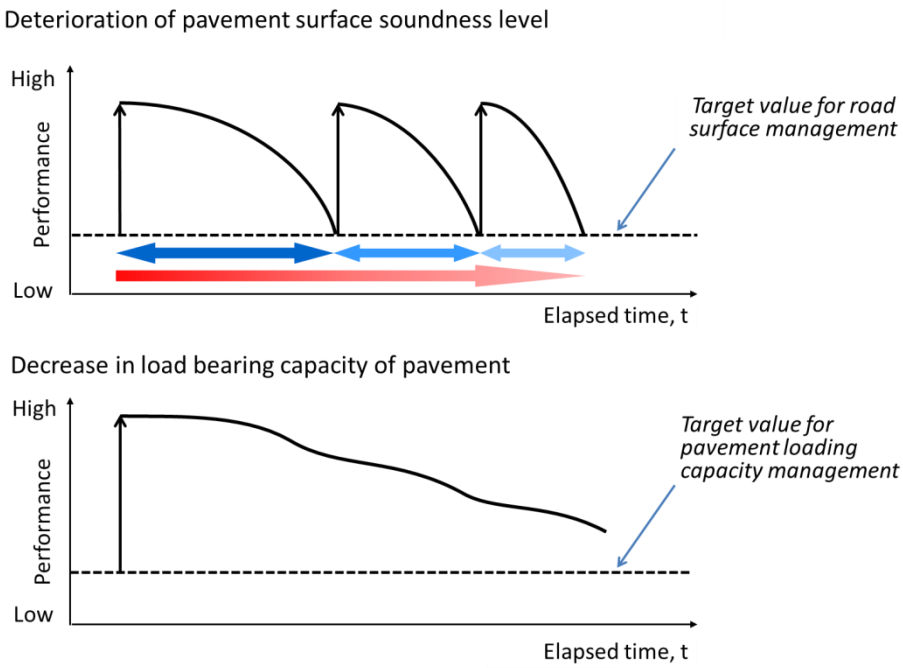


Figure 3.1 Compound deterioration process of pavement

In this study, it is considered that the decrease in pavement load bearing capacity takes negative effects to accelerate its surface deterioration. Additionally, we also consider that the decrease in the soundness level of the road surface also decreases pavement load bearing capacity. In other words, it is considered that the deterioration of the road surface soundness level and the decrease in load bearing capacity have a compound mutual interaction that makes the speed of deterioration and decrease influence each other. Figure 3.1 shows a typical compound deterioration process of pavement. The upper graph shows the road surface deterioration process, and the lower graph shows the deterioration process of the overall pavement load bearing capacity. In this figure, the deterioration process of the road surface

appears to proceed more rapidly than that of the overall pavement load bearing capacity. In actual road management operation, experience indicates that when the load bearing capacity of the overall pavement decreases, the road surface deterioration proceeds more rapidly. Additionally, there is a possibility that when the road surface soundness level decreases, the decrease in the load bearing capacity is accelerated. In this study, we consider that the deterioration process of the load bearing capacity is relatively slower than that of the road surface, and the deterioration speed of the road surface is accelerated according to the reduction of load bearing capacity; simultaneously, the decrease in the load bearing capacity is also accelerated according to the decrease in soundness level of the surface layer.

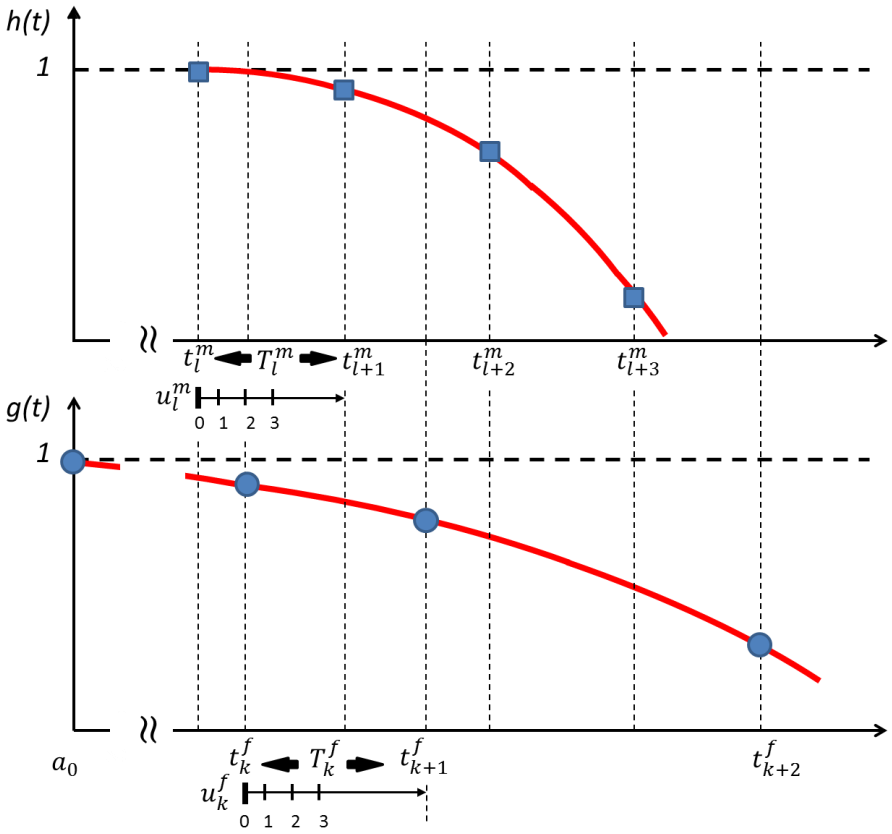
3.2.3 Systematic Missing Data

In order to manage the road, it is necessary to grasp not only the current condition of pavements but also its deterioration in the future. However, for more accurate image of pavement current conditions and the forecasted deterioration, it is crucial to prepare sufficient volume of reliable deterioration information, which may lead to increase survey expenses especially in the cases of huge road networks. Under budgetary restrictions and other constraints, many road administrators often select a proper survey scheme based on their experiences. For example, when it is determined that the road surface soundness level has decreased rapidly since the previous repair, there is highly possibility that the pavement strength deterioration is progressing. In this case, FWD survey should be set higher priority for implementation instead of road condition survey to determine solutions on pavement structure. Benchmarking evaluation using available time-series datasets of pavement surface conditions should be an useful tool to specify the target sections for structural investigation.

In other cases, when we found that the pavement surface conditions are quite stable in comparison with the data in previous surveys, and the overall pavement structure is sound, only regular pavement condition survey should be conducted instead of intensive tests for pavement strength. In reality, to implement structural survey like FWD test, negative influence to traffic cannot be avoided that generates social costs due to slow moving, congestion even poor traffic safety. The more wider the survey scope, the more serious influence to traffic. Regarding this practical issue, there should have a strategy to optimize pavement surveys taking the constraints of budget and traffic disturbance into account. Moreover, formulation deterioration forecast model with a time lag between road surface conditions and FWD surveys is also crucial.

In this study, we discuss the problem of the systematic absence of partial data required for estimating the compound Markov deterioration model, which is generated by asynchronous FWD and road surface condition surveys. When a road surface condition survey is conducted, FWD survey may not be performed. In another case, when pavement surface deteriorates too rapidly or abnormally detected by visual inspection or road patrol, FWD survey shall be

conducted independently with road surface surveys for determination of pavement repair solutions. In reality, even when we cannot simultaneously observe data on the road surface soundness level and load bearing capacity, it is possible to simulate a compound deterioration process with partial information if either data is available. To simulate the deterioration process of the road surface, data on the load bearing capacity is required. Although we cannot obtain information on the road bearing capacity at the time of observing the road surface soundness level, if we can obtain the most recent data on the load bearing capacity, we can estimate that the current load bearing capacity is equal to or lower than that (complementary information 1). On the other hand, when we observe the load bearing capacity, we can estimate that the current road surface soundness level is equal to or lower than that of the most recent observation (complementary information 2). With such partial information, we can improve the estimation accuracy of the compound Markov deterioration model.



Note) The upper graph shows the typical deterioration process of the road surface soundness level, and the lower graph shows the typical deterioration process of the pavement load bearing capacity. Each ■ mark indicates the time point of the road surface condition survey, and ● indicates the time point of the FWD survey. At time point t_l^m, the road surface condition survey is conducted, and m local time point axis u_l^m, of which the starting point is time point t_l^m is defined. Similarly, at f local time point t_k^f, the load bearing capacity survey is conducted, and f local time point axis u_k^f, of which the starting point is time point t_k^f, is defined.

Figure 3.2 Compound deterioration process of pavement and local time axis

Thus, we use two types of complementary information in this study and propose an estimation method for the compound Markov deterioration model. In the following section, we first

assume a case in which we can obtain full information and formulate the compound Markov deterioration model. Then, in Section 4, we propose the compound hidden Markov deterioration model that demonstratively considers the systematic absence of inspection data.

3.3 COMPOUND MARKOV DETERIORATION MODEL

3.3.1 Prerequisite for Modeling

We consider that a road manager opens a road for traffic after completion of new construction or renewal at calendar time a_0 as shown in Figure 3.2. We introduce $t = 0, 1, 2, \dots, T$, which is a discrete time axis at which a_0 is the initial time point, $t = 0$; T is the end time of the observation period. We designate a point on the discrete time axis as "a time point" to distinguish it from the calendar time. The target deterioration process consists of two compound deterioration processes, those of the road surface soundness level and the load bearing capacity. To simplify this case, we consider that pavement renewal has not been conducted since the initial time point. Once the pavement renewal is conducted, we consider that the calendar time is the initial time point. As shown in Figure 3.2, a pavement condition survey is conducted at the time points on the discrete time axis, such as $0,\ t_1^m, \dots, t_l^m, \dots$. We introduce the local discrete time axis, $u_l^m = 0, 1, 2, \dots, T_l^m$, where time point t_l^m at which m time of the road condition survey is conducted is the starting point $u_l^m = 0$. Here, T_l^m is a time length or time interval from the pavement condition survey at time point t_l^m till the next pavement condition survey, and expressed as $T_l^m = t_{l+1}^m - t_l^m$. The time point on the local discrete time axis, u_l^m, is called the m local time point. Similarly, at the time points on the discrete time axis, $0,\ t_1^f, \dots, t_k^f, \dots$, an FWD survey is conducted. Furthermore, we introduce the local discrete time axis, $u_k^f = 0, 1, 2, \dots, T_k^f$, where time point t_k^f at which an FWD survey is conducted is the starting point, $u_k^f = 0$. Here, T_k^f is a time length or interval from the FWD survey at time point t_k^f till the next FWD survey, and expressed as $T_k^f = t_{k+1}^f - t_k^f$. The time point on the discrete time axis, u_k^f, is called the f local time point. Time point t_l^m $(l = 0, 1, \dots, N_m)$, at which the road condition survey is conducted, and time point t_k^f $(k = 0, 1, \dots, N_f)$ do not always match, except for the time to examine pavement conditions on final acceptance work before handing over for operation. Here, we introduce a map to express the correspondence between the f and m local time points, as follows:

$$\omega^f(u_l^m) = u_k^f \tag{3.1a}$$

$$\omega^m(u_k^f) = u_l^m \tag{3.1b}$$

We express the pavement load bearing capacity at the *f* local time point (u_k^f) with a discrete state variable, $g(u_k^f) = s\ (s = 1, \dots, S\ ;\ u_k^f = 0, \dots, T_k^f)$. The rating $s\ (s = 1, \dots, S)$ indicates that the larger s , the lower the load bearing capacity. The case $g(u_k^f) = S$ shows that the load bearing capacity reaches the target limit of requirement on pavement strength. At initial time point $t_0^f = 0, g(0) = 1$. Then, we describe the road surface soundness level at m local time point u_l^m with I ratings $h(u_l^m) = i\ (i = 1, \dots I\ ;\ u_l^m = 0, \dots, T_l^m)$. At initial time point $t_0^m = 0$, the formula $h(0) = 1$ is formed. $h(u_l^m) = I$ shows another target limit of the road surface. Pavement repair focusing on road surface may repeat several times during the target period. Although no repair time point of the pavement surface is clearly described, we consider that a road condition survey is conducted at each repair time point, and that the road surface soundness recovers to level 1. In this study, we individually express the road surface deterioration process and the decrease of the pavement load bearing capacity with Markov chain models. The two Markov chain models have a mutual interaction.

3.3.2 Deterioration Process of Load Bearing Capacity

At time point $t_0^f(u_0^f) = 0$, when the service (or renewal) of the road starts, the load bearing capacity of the pavement is fixed at $g(0) = 1$. We express the transition of the load bearing capacity deterioration generated between the *f* local time points from u_k^f to $(u_k^f + 1)$ with Markov transition probability. The length of unit periods $[\,u_k^f\,,\ u_k^f + 1)$ is standardized at 1. The soundness level, *s*, at the *f* local time point, $u_k^f \neq 0$, cannot be observed, but is currently considered to be known. The Markov transition probability showing the road surface deterioration process at local period $[\,u_k^f\,,\ u_k^f + 1)$, period on the discrete time axis $[\,t_k^f + u_k^f\,,\ t_k^f + u_k^f + 1)$, has the load bearing capacity $g(\,u_k^f) = 1$, evaluated at the *f* local time point, u_k^f (time point $t_k + u_k$), and the road surface soundness level, $h(\omega^m(u_k^f)) = i$, at local time point m $\omega^m(u_k^f)$, corresponding to the f local time point u_k^f as given conditions. This can be defined as a conditional probability where the soundness level, $g(u_k^f + 1) = v$, is generated at the next *f* local time point, $(u_k^f + 1)$, as

$$Prob\left[g(u_k^f + 1) = v \middle| g(u_k^f) = s, h(\omega^m(u_k^f)) = i\right] = p^{sv}(i) \tag{3.2}$$

We standardize the time length as 1. The Markov transition probability can be expressed with the Markov deterioration hazard model developed by Tsuda et al.[6] The hazard rate, $\lambda^s(i)$ [3], of the load bearing capacity, $s\ (s = 1, \dots, S - 1)$, at time point t of which the given condition is the soundness level, i (hereafter called the load bearing capacity hazard rate), is expressed as follows:

$$\lambda^s(i) = \beta_0^i x \beta^s = \beta_0^i \lambda^s \tag{3.3}$$

$\beta_0^i\ (i = 1, \dots, I - 1)$ is a scale parameter showing that the heterogeneity of the load bearing capacity deterioration speed, depending on road surface soundness level i, $x = (x_1, \dots, x_Q)$, is an explanatory variable vector, and $\beta^s = (\beta_1^s, \dots, \beta_Q^s)'$ is an unknown parameter vector.

The mark $'$ indicates transposition, and Q indicates the number of explanatory variables. We standardize to $\beta_0^1 = 1$. At this time, the probability of load bearing capacity s at f local time point u_k^f under soundness level i, and the continuation of s at f local time point $(u_k^f + 1)$, is expressed as follows:

$$p^{ss}(i) = exp\,\{-\lambda^s(i)\} = exp\,\{-\beta_0^i \lambda^s\} \tag{3.4}$$

Further, the Markov transition probability, $p^{sv}(i)$ $(s = 1, \dots, S-1\,;\ v = s+1, \dots, S)$, of the transition of the load bearing capacity from s to v $(> s)$ between f local time points from u_k^f and $(u_k^f + 1)$ is expressed as follows:

$$p^{sv}(i) = \sum_{m=s}^{v} \prod_{z=s}^{m-1} \frac{\lambda^z}{\lambda^z(i) - \lambda^m(i)}$$

$$\cdot \prod_{z=m}^{v-1} \frac{\lambda^z(i)}{\lambda^{z+1}(i) - \lambda^m(i)} \exp\{-\lambda^m(i)\}$$

$$(s = 1, \dots, S-1\,;\ v = s+1, \dots, S) \tag{3.5}$$

We consider that the following notational rules are formed [6]:

$$\begin{cases} \prod_{z=s}^{m-1} \frac{\lambda^z}{\lambda^z(i) - \lambda^m(i)} = 1 & (m = s) \\ \prod_{z=m}^{v-1} \frac{\lambda^z(i)}{\lambda^{z+1}(i) - \lambda^m(i)} = 1 & (m = v) \end{cases}$$

These rules are simplified as shown below:

$$\prod_{z=s,\neq m}^{v-1} \frac{\lambda^z(i)}{\lambda^z(i) - \lambda^m(i)} \exp\{-\lambda^m(i)\}$$

$$\prod_{z=s}^{m-1} \frac{\lambda^z(i)}{\lambda^z(i) - \lambda^m(i)} \prod_{z=m}^{v-1} \frac{\lambda^z(i)}{\lambda^{z+1}(i) - \lambda^m(i)}\ exp\{-\lambda^m(i)\}$$

Additionally, $p^{sS}(i)$ can be expressed as follows based on the Markov transition probability conditions:

$$p^{sS}(i) = 1 - \sum_{v=s}^{S-1} p^{sv}(i) \quad (s = 1, \dots, S-1) \tag{3.6}$$

With formula (3.6), the transition probability matrix in period $[\,u_k^f\,,\ u_k^f + 1)$ can be defined as follows:

$$\boldsymbol{P}(\boldsymbol{i}) = \begin{pmatrix} p^{11}(i) & \cdots & p^{1S}(i) \\ \vdots & \ddots & \vdots \\ 0 & \cdots & p^{SS}(i) \end{pmatrix} \tag{3.7}$$

3.3.3 Road Surface Deterioration Process

The transition of the road surface soundness level generated between the m local time point from u_l^m to $u_l^m + 1$ is expressed by using the Markov transition probability. We standardize the time length of unit period $[u_l^m, u_l^m + 1)$ as 1. The load bearing capacity, s, at the m local time point, u_l^m, cannot be observed, but is currently considered to be known. The Markov transition probability showing the road surface deterioration process at local period $[u_l^m, u_l^m + 1)$ (period on discrete time axis $[t_l^m + u_l^m, t_l^m + u_l^m + 1))$ uses the given conditions of load bearing capacity $g(\omega^f(u_l^m)) = s$ and road surface soundness level $h(u_l^m) = i$, evaluated at u_l^m, and can be defined as a conditional probability in which the soundness level, $h(u_l^m + 1) = j$, is generated at the next local time point m $(u_l^m + 1)$, as shown below:

$$Prob\left[h(u_l^m + 1) = j | h(u_l^m) = i, g(\omega^f(u_l^m)) = s\right] = \pi^{ij}(s) \tag{3.8}$$

The hazard rate, $\mu^i(s)$, of the soundness level, i (i =1, $\cdot\cdot\cdot$, $I-1$), with the load bearing capacity, s, as the given condition is expressed as follows:

$$\mu^i(s) = \gamma_0^s \boldsymbol{y}\gamma^i = \gamma_0^s \mu^i \tag{3.9}$$

Here, γ_0^s $(s = 1, \dots, S-1)$ is a scale parameter that shows the heterogeneity of the road surface deterioration speed, depending on s, $\boldsymbol{y} = (y^1, \dots, y^V)$ is an explanatory variable vector, $\boldsymbol{\gamma^i} = (\gamma_1^i, \dots, \gamma_V^i)'$ is an unknown parameter vector, and $\mu^i = \boldsymbol{y}\gamma^i$. We standardize as $\gamma_0^1 = 1$. At this time, the probability of the soundness level, i, under s at u_l^m, and continuation of i at the time point m $(u_l^m + 1)$, are expressed as follows:

$$\pi^{ii}(s) = exp\{-\mu^i(s)\} = exp\{-\gamma_0^s \mu^i\} \tag{3.10}$$

Further, the Markov transition probability, $\pi^{ij}(s)$ $(i = 1, \dots, I-1\,;\, j = i+1, \dots, I)$, of the transition of the soundness level from i to j (>i) between u_l^m and $(u_l^m + 1)$ is expressed as follows:

$$\pi^{ii}(s) = \sum_{z=i}^{j} \prod_{r=i \neq z}^{j-1} \frac{\mu^r(s)}{\mu^r(s) - \mu^z(s)} exp\{-\mu^z(s)\}$$

$$(i = 1, \dots, I-1\,;\, j = i+1, \dots, I) \tag{3.11}$$

Additionally, $\pi^{iI}(s)$ can be expressed as follows based on the Markov transition probability conditions:

$$\pi^{iI}(s) = 1 - \sum_{j=i}^{I-1} \pi^{ii}(s) \qquad (s = 1, \dots, S-1) \tag{3.12}$$

With the previously mentioned transition probabilities, we can define the Markov transition probability matrix determining each element as conditional probability (3.8), defined in local period $[u_l^m,\ u_l^m + 1)$ as follows:

$$\Pi(s) = \begin{pmatrix} \pi^{11}(s) & \cdots & \pi^{1I}(s) \\ \vdots & \ddots & \vdots \\ 0 & \cdots & \pi^{II}(s) \end{pmatrix} \tag{3.13}$$

3.3.4 Compound Markov Deterioration Model

We consider that the pavement is renewed at the initial time point, $t = 0$, the load bearing capacity is fixed as $g(0) = 1$, and the soundness level is fixed as $h(0) = 1$. Next, as time passes, the road surface deterioration and decrease in load bearing capacity proceed. We describe the compound deterioration condition of the pavement, $x\ (x = 1, \dots, X)$, as $x(t) = \{\tilde{h}(t),\ \tilde{g}(t)\}$ with both the road surface soundness level, $\tilde{h}(t)$, and the load bearing capacity, $\tilde{g}(t)$, at time point t. The road surface soundness level, $\tilde{h}(t)$, and load bearing capacity, $\tilde{g}(t)$, are defined with time point t on the discrete time axis, and "$\sim$" is used to discriminate them from the road surface soundness level, $h(u_l^m)$, and load bearing capacity, $g(u_k^f)$, defined with m and f local time points. Additionally, $X = I \times S$. Further, the compound deterioration condition, $x(t) = x\ (x = 1, \dots, X)$ at t corresponds to $(1,1), \dots, (1,S), (2,1), \dots, (2,S), (3,1), \dots, (I,S)$. When the compound deterioration condition, $x(t) = \{\tilde{h}(t),\ \tilde{g}(t)\}$, takes the condition variable $x = (i, s)$, the marks specifying the $\tilde{h}(t)$, $\tilde{g}(t)$ elements of the compound deterioration condition, $x(t)$, are described as $\tilde{h}_x(t) = i,\ \tilde{g}_x(t) = s$. The frequency distribution of the compound deterioration condition is expressed as $v(t) = \{v_1(t), \dots, v_X(t)\}$. The frequency distribution at the initial time point is $v(0) = (1, 0, \dots, 0)$. We define the transition probability matrix, Ω, in the compound deterioration condition as follows:

$$\Omega = \begin{pmatrix} \omega_{11} & \cdots & \omega_{1X} \\ \vdots & \ddots & \vdots \\ \omega_{X1} & \cdots & \omega_{XX} \end{pmatrix}$$

$$= \begin{pmatrix} \omega_{11}^{11} & \omega_{11}^{12} & \dots & \omega_{11}^{j\vartheta} & \dots & \omega_{11}^{IS} \\ \vdots & \vdots & \ddots & \vdots & \dots & \vdots \\ \omega_{is}^{11} & \omega_{is}^{12} & \dots & \omega_{is}^{j\vartheta} & \dots & \omega_{is}^{IS} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ \omega_{IS}^{11} & \omega_{IS}^{12} & \dots & \omega_{IS}^{j\vartheta} & \dots & \omega_{IS}^{IS} \end{pmatrix} \tag{3.14}$$

Element ω_{is}^{jv} $(i, j = 1, \dots, I\ ;\ s, v = 1, \dots, S)$ is defined as follows:

$$\omega_{is}^{jv} = p^{sv}(i)\pi^{ii}(s) \tag{3.15}$$

Additionally, the formula $\omega_{is}^{jv} = 0$ (when either or both $i > j$ and $s > v$ is applicable). In this case, unless the pavement is renewed, the compound deterioration process is expressed with the following Markov chain:

$$\boldsymbol{v}(t) = \boldsymbol{v}(0)\{\boldsymbol{\Omega}\}^t \tag{3.16}$$

3.4 COMPOUND HIDDEN MARKOV DETERIORATION MODEL

3.4.1 Survey Scheme

With a certain road section as a target, we consider a mechanism of the observation of data related to the pavement deterioration condition, collected in road condition or FWD surveys. Without damaging generality, we consider that no road surface repair is conducted during the target period. If road surface repair is conducted, we can consider that road surface sound level 1 is observed at that time point. A road condition survey is conducted at time point $\bar{t}_1^m, \dots, \bar{t}_{N_m}^m$ on the discrete time axis, and the road surface soundness level $\tilde{h}(\bar{t}_l^m)$ $(l = 0, \dots, N_m)$ is observed. Similarly, FWD survey is conducted at time point $\bar{t}_1^f, \dots, \bar{t}_{N_f}^f$, and the pavement load bearing capacity $\tilde{g}(\bar{t}_k^f)$ $(k = 0, \dots, N_f)$ is observed. The road surface condition and FWD surveys are not conducted simultaneously. The collective data in the road condition and FWD surveys are described as $\tilde{\Xi} = \left[\{\bar{t}_k^f, \tilde{g}(\bar{t}_k^f)\ \ (k = 0, \dots, N_f)\},\ \{\bar{t}_l^m, \tilde{h}(\bar{t}_l^m)\ \ (l = 0, \dots, N_m)\}\right]$. As previously mentioned, we individually define the starting points of *m* and *f* local time points as the time points when the road condition and FWD surveys are conducted. The correspondence between the m and f local time points is defined with formulas (3.1a) and (3.1b). Here, to make the description more useful, we re-sort the time points at which the road condition and FWD surveys are conducted along with the time axis in order of calendar time, and newly define time point τ_n $(n = 0, 1, \dots, N)$. Additionally, $N = N_m + Nf$. At time point τ_n $(n = 0, \dots, N)$ on the discrete time axis, at least either the road condition survey or the FWD survey will be conducted. A group of time points at which the road condition surveys are conducted is described as ρ^m, and that at which the FWD surveys are conducted is described as ρ^f. At time point τ_n $(n = 0, \dots, N)$, only survey results of either the road condition survey or the FWD survey can be acquired. A type of survey conducted at time point τ_n is expressed as $q(\tau_n)$:

$$q(\tau_n) = \begin{cases} m & Road\ surface\ condition\ survey \\ f & FWD\ survey \end{cases} \tag{3.17}$$

The condition variable, $r(\tau_n)$, acquired at τ_n is expressed as follows:

$$r(\tau_n) = \begin{cases} \tilde{h}(\tau_n) & \tau_n \in \rho^m \\ \tilde{g}(\tau_n) & \tau_n \in \rho^f \end{cases} \tag{3.18}$$

At this time, a group of the observation data defined with time point τ_n $(n = 0, \dots, N)$ can be expressed as $\Xi = \{\tau_n, q(\tau_n), r(\tau_n),\ (n = 0, \dots, N)\}$.

3.4.2 Data Observation Process

When the compound deterioration process of the pavement proceeds according to formula (3.16), we derive the probability of observation of inspection data Ξ (likelihood). Let us consider a case in which the initial time point is t_0. The road surface soundness level at the initial time point is expressed as $\tilde{h}(t_0) = 1$, and the load bearing capacity is expressed as $\tilde{g}(t_0) = 1$. The frequency distribution vector of the compound deterioration condition is $v(0) = (1,0, \dots, 0)$. Let us consider another case in which the time point is τ_1, which belongs to group ρ^m. In other words, at time point $\tau_1 \in \rho^m$, a road condition survey is conducted and the road surface soundness level $\tilde{h}(\tau_1) = i$ is observed. In period $[\tau_0, \tau_1)$, the compound deterioration process of the pavement proceeds according to formula (3.16). When the time length is defined as $\Delta_0 = \tau_1 - \tau_0$, the frequency distribution of the compound deterioration condition at time point τ_1 is expressed based on formula (3.16) as follows:

$$\boldsymbol{v}(\tau_1) = \boldsymbol{v}(0)\{\boldsymbol{\Omega}\}^{\Delta_0} \tag{3.19}$$

At τ_1, with the observation result, the road surface soundness level is fixed as $\tilde{h}(\tau_1) = \bar{i}$. According to this information, we define an occurrence frequency, $\tilde{v}_x(\tau_1)$, of the compound deterioration condition, $x(\tau_1) = (\bar{i}, s)$, as follows:

$$\tilde{v}_x(\tau_1) = \begin{cases} 0 & i \neq \bar{i} \\ \dfrac{v_x(\tau_1)}{\sum_{y \in G(\bar{i})} v_y(\tau_1)} & i = \bar{i} \end{cases} \tag{3.20}$$

Group $G(\bar{i})$ is defined as $G(\bar{i}) = \{y | y = (\bar{i}, s),\ (s = 1, \dots, S)\}$. Next, we consider a case in which τ_1 belongs to group ρ^f. At time point τ_1, the road surface soundness level is fixed as $\tilde{g}(\tau_1) = \bar{s}$. According to this information, we define an occurrence frequency, $\tilde{v}_x(\tau_1)$, of the compound deterioration condition, $x(\tau_1) = (i, \bar{s})$, as follows:

$$\tilde{v}_x(\tau_1) = \begin{cases} 0 & s \neq \bar{s} \\ \dfrac{v_x(\tau_1)}{\sum_{y \in G(\bar{s})} v_y(\tau_1)} & s = \bar{s} \end{cases} \tag{3.21}$$

Group $G(\bar{s})$ is defined as $G(\bar{s}) = \{y | y = (i, \bar{s}),\ (i = 1, \dots, I)\}$. In this case, the frequency distribution of the compound deterioration condition at time point τ_2 is defined as follows:

$$\boldsymbol{v}(\tau_2) = \tilde{\boldsymbol{v}}(\tau_1)\{\boldsymbol{\Omega}\}^{\Delta_1} \tag{3.22}$$

By generalizing the preceding discussion, we can express the frequency distribution of the compound deterioration condition, $x(\tau_n) = (i, s)$, for the time when the observation data at time point τ_n is acquired as follows:

In the case of $(\tau_n \in \rho^m)$

$$\tilde{v}_x(\tau_n) = \begin{cases} 0 & i \neq \bar{i} \\ \dfrac{v_x(\tau_n)}{\sum_{y \in G_{\bar{i}}(\tau_n)} v_y(\tau_n)} & i = \bar{i} \end{cases} \tag{3.23a}$$

In the case of $(\tau_n \in \rho^f)$

$$\tilde{v}_x(\tau_n) = \begin{cases} 0 & s \neq \bar{s} \\ \dfrac{v_x(\tau_n)}{\sum_{y \in G_{\bar{s}}(\tau_n)} v_y(\tau_n)} & s = \bar{s} \end{cases} \tag{3.23b}$$

In this case, the frequency distribution of the compound deterioration condition at time point τ_{n+1} can be expressed as follows:

$$\boldsymbol{v}(\tau_{n+1}) = \tilde{\boldsymbol{v}}(\tau_n)\{\boldsymbol{\Omega}\}^{\Delta_n} \tag{3.24}$$

3.4.3 Likelihood Function

We consider that we can acquire observation data Ξ through the overall period and express these data acquired from the initial time point to time point τ_n $(0 < n \leq N)$ as $\xi_n = \{\tau_a, q(\tau_a), r(\tau_a),\ (a = 0, \dots, n)\}$. Here, we define a dummy variable specifying the data observed at τ_n as follows:

$$\delta(\tau_n) = \begin{cases} 1 & r(\tau_k) = \bar{i}\ (\tau_k \in \rho^m) \\ 1 & r(\tau_k) = \bar{s}\ (\tau_k \in \rho^k) \\ 0 & Others \end{cases} \tag{3.25}$$

The probability, $\ell(\xi_1)$, of the observation of data, ξ_1, before τ_1 is as follows:

$$\ell_1(\xi_1) = \sum_{x=1}^{X} \{v_x(\tau_1)\}^{\delta(\tau_1)} \tag{3.26}$$

Concerning τ_2 and later, the following formulas are recursively defined:

$$\ell_2(\xi_2) = \ell_1(\xi_1) \sum_{x=1}^{X} \{v_x(\tau_2)\}^{\delta(\tau_2)} \tag{3.27a}$$

$$\vdots$$

$$\ell_N(\xi_N) = \ell_{N-1}(\xi_{N-1}) \sum_{x=1}^{X} \{v_x(\tau_N)\}^{\delta(\tau_N)} \tag{3.27b}$$

In this case, the likelihood of the observation of information group Ξ can be expressed as follows:

$$\mathcal{L}(\Xi : \theta) = \prod_{n=1}^{N} \sum_{x=1}^{X} \{v_x(\tau_N : \theta)\}^{\delta(\tau_n)} \tag{3.28a}$$

$$\boldsymbol{v}(\tau_{n+1}) = \tilde{\boldsymbol{v}}(\tau_n)\{\boldsymbol{\Omega}\}^{\Delta_n} \quad (n = 1, \dots, N) \tag{3.28b}$$

Here, $\theta = \{\beta_0^s, \beta^s, \gamma_0^i, \gamma^i \ : \ s = 1, \dots, S-1, \ i = 1, \dots, I-1\}$ is an unknown parameter vector. To demonstratively express that index hazard models (3.3) and (3.9), which express transition probability ω_{is}^{jv}, depend on parameter θ, we describe $\tilde{v}_x(\tau_n : \theta)$. The likelihood function of compound hidden Markov deterioration model (3.28a) is a high-degree nonlinear polynomial equation concerning θ, and optimizing conditions at the first stage have a significant amount of solutions (including complex number solutions)[16],[17]. It is necessary to select a real root between 0 and 1 for an estimated figure of transition probability ω_{is}^{jv}. If we use Bayesian estimation instead of the method of maximum likelihood, we can avoid solving a high-degree nonlinear polynomial equation. However, this case unfortunately requires a significantly vast volume of calculation because likelihood functions (3.28a) and (3.28b) include many sections [16]-[18]. To eliminate this calculation problem, a completion operation of the likelihood function is required.

3.4.4 Completion Operation

In order to conduct the completion operation of the likelihood function, a latent variable is defined. For simple description, use the m and f local time axes. It is considered that the observation data, $\Xi = \{\bar{\tau}_n, \bar{q}(\bar{\tau}_n), \bar{r}(\bar{\tau}_n), (n = 0, \dots, N)\}$, can be acquired through the overall period. The symbol "$\bar{\ }$" indicates that the indicated value is the actual measurement value. Period $[\bar{\tau}_n, \bar{\tau}_{n+1})$ can be described with these two types of local time axes. We consider that a road condition survey is conducted at time point $\bar{\tau}_n$; and each time point on the m and f local time axes at time point $\bar{\tau}_n - 1$ is expressed as $u_l^m - 1$ and $u_k^f - 1$. In this case, each time point on the m and f local time axes in period $[\bar{\tau}_n, \bar{\tau}_{n+1})$ is expressed as follows:

$$0, 1, \dots, \dots, \Delta_n \tag{3.29a}$$

$$u_k^f, u_k^f + 1, \dots, u_k^f + \Delta_n \tag{3.29b}$$

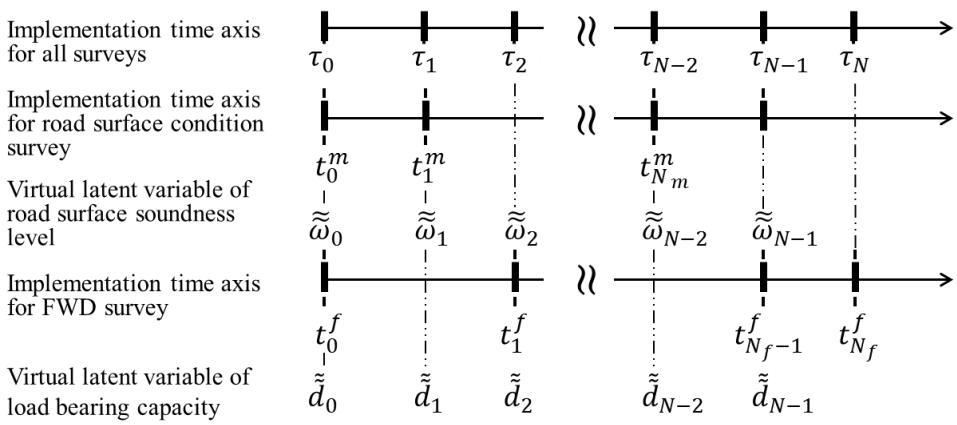


Figure 3.3 Virtual latent variable matrix

On the other hand, when an FWD survey is conducted at time point $\bar{\tau}_n$, each time point on the m and f local time axes in period $[\bar{\tau}_{n,}\ \bar{\tau}_{n+1})$ transitions as follows:

$$u_l^m, u_l^m + 1, \dots, u_l^m + \Delta_n \tag{3.30a}$$

$$0, 1, \dots, \Delta_n \tag{3.30b}$$

Here, a transition pattern of the road surface soundness level in period $[\bar{\tau}_{n,}\ \bar{\tau}_{n+1})$ is expressed with the following latent variable:

$$\omega_n = \begin{cases} (\omega_0, \dots, \omega_{\Delta_n} & \bar{q}(\bar{\tau}_n) = m \\ (\omega_{u_l^m}, \dots, \omega_{u_l^m + \Delta_n} & \bar{q}(\bar{\tau}_n) = f \end{cases} \tag{3.31}$$

Additionally, we express a transition pattern of the load bearing capacity with the following latent variable:

$$d_n = \begin{cases} (d_{u_k^f}, \dots, d_{u_k^f + \Delta_n} & \bar{q}(\bar{\tau}_n) = m \\ (d_0, \dots, d_{\Delta_n} & \bar{q}(\bar{\tau}_n) = f \end{cases} \tag{3.32}$$

In the case of $\bar{q}(\bar{\tau}_n) = m$, the formula $\omega_0 = \tilde{h}(\bar{\tau}_n)$ is applicable, and in case of $\bar{q}(\bar{\tau}_n) = f$, the formula $d_0 = \tilde{g}(\bar{\tau}_n)$ is applicable.

Due to the deterioration process characteristics, unless the pavement is renewed, the deterioration of the road surface soundness level and decrease in the load bearing capacity proceed with time. In this case, between the latent variables, the following conditions are satisfied:

$$\bar{\omega}_0 \leq \cdots \leq \omega_{u_l^m} \leq \cdots \leq \omega_{T_l^m - 1} \leq \bar{\omega}_{T_l^m} \tag{3.33a}$$

$$\bar{d}_0 \leq \cdots \leq d_{u_k^f} \leq \cdots \leq d_{T_k^f - 1} \leq \bar{d}_{T_k^f} \tag{3.33b}$$

Here, $\bar{\omega}_0 = \tilde{h}(t_l^m), \bar{\omega}_{T_l^m} = \tilde{h}(T_l^m)$ and $\bar{d}_0 = \tilde{g}\left(t_k^f\right), \bar{d}_{T_k^f} = \tilde{g}\left(T_k^f\right)$. Elements of the true soundness level vector, ω_n, and the load bearing capacity vector, d_n, are not observed individually, except for $\bar{\omega}_0, \bar{\omega}_{T_l^m}$ and $\bar{d}_0, \bar{d}_{T_k^f}$, but we assume that these latent variables can be measured. In this case, to simplify the discussion, we describe the virtual latent variable vector matrix over the total target period, as shown in Figure 3.3, and add subscripts from time points $\tau_0 = 0$ to τ_{N-1}. With these descriptions, we obtain the following latent variable vector matrix:

$$\tilde{\bar{\omega}}_0, \dots, \tilde{\bar{\omega}}_n, \dots, \tilde{\bar{\omega}}_{N-1} \tag{3.34a}$$

$$\tilde{\bar{d}}_0, \dots, \tilde{\bar{d}}_n, \dots, \tilde{\bar{d}}_{N-1} \tag{3.34b}$$

The virtual latent variables $\tilde{\omega} = \{\tilde{\bar{\omega}}_n,\ (n = 0, \dots, N-1)\}, \tilde{d} = \left\{\tilde{\bar{d}}_n,\ (n = 0, \dots, N-1)\right\}$ are given. In this case, likelihood $\ell(\tilde{\bar{\omega}}_1, \tilde{\bar{d}}_1, \bar{\xi}_1)$ completed with virtual latent variable vectors $\tilde{\bar{\omega}}_1, \tilde{\bar{d}}_1$ at time point τ_1 is expressed as follows:

$$\tilde{\ell}_1\left(\tilde{\bar{\omega}}_1, \tilde{\bar{d}}_1, \bar{\xi}_1\right) = \tilde{\bar{v}}_{\tilde{\bar{\omega}}_{\tau_1}, \tilde{\bar{d}}_{\tau_1}}(\tau_1) = \prod_{y_0=0}^{T_{0-1}} \omega_{\tilde{\bar{\omega}}_{y_0}\tilde{\bar{d}}_{y_0}}^{\tilde{\bar{\omega}}_{y_0+1}\tilde{\bar{d}}_{y_0+1}} \tag{3.35}$$

Concerning τ_2 and later, the following formulas are recursively defined:

$$\tilde{\ell}_2\left(\tilde{\bar{\omega}}_1, \tilde{\bar{d}}_1, \bar{\xi}_2\right) = \tilde{\ell}_1\left(\tilde{\bar{\omega}}_1, \tilde{\bar{d}}_1, \bar{\xi}_1\right)\tilde{\bar{v}}_{\tilde{\bar{\omega}}_{\tau_2}, \tilde{\bar{d}}_{\tau_2}}(\tau_2) \tag{3.36a}$$

$$\vdots$$

$$\tilde{\ell}_N\left(\tilde{\bar{\omega}}_N, \tilde{\bar{d}}_N, \bar{\xi}_N\right) = \tilde{\ell}_{N-1}\left(\tilde{\bar{\omega}}_{N-1}, \tilde{\bar{d}}_{N-1}, \bar{\xi}_{N-1}\right)\tilde{\bar{v}}_{\tilde{\bar{\omega}}_{\tau_N}, \tilde{\bar{d}}_{\tau_N}}(\tau_N) \tag{3.36b}$$

The formula below is applicable:

$$\tilde{\bar{v}}_{\tilde{\bar{\omega}}_{\tau_{n+1}}, \tilde{\bar{d}}_{\tau_{n+1}}}(\tau_{n+1}) = \prod_{y_n=0}^{T_{n-1}} \omega_{\tilde{\bar{\omega}}_{y_n}\tilde{\bar{d}}_{y_n}}^{\tilde{\bar{\omega}}_{y_n+1}\tilde{\bar{d}}_{y_n+1}} \tag{3.37}$$

In this case, the likelihood of observation of information group Ξ is defined as follows:

$$\tilde{\mathcal{L}}\left(\tilde{\bar{\omega}}, \tilde{\bar{d}}, \bar{\Xi} : \theta\right) = \prod_{n=0}^{N-1}\prod_{y_n=0}^{T_{n-1}} \omega_{\tilde{\bar{\omega}}_{y_n}\tilde{\bar{d}}_{y_n}}^{\tilde{\bar{\omega}}_{y_n+1}\tilde{\bar{d}}_{y_n+1}} \tag{3.38}$$

The operation mentioned above is called "completion". Completed likelihood function (3.38) is much more simplified than normal likelihood functions (3.28a) and (3.28b). However, latent variables $\tilde{\tilde{\omega}}$ and $\tilde{\tilde{d}}$ included in completed likelihood function (3.38) are unmeasurable. We can derive a full conditional posterior distribution of variables $\tilde{\omega}$ and $\tilde{d}$ by deploying completed likelihood function (3.38).

3.4.5 Probability Distribution of Latent Variable

Owing to the deterioration property of the road surface soundness level, condition (3.33a) is established unless a repair is conducted. Here, when we assume with the latent variables that $\tilde{\tilde{\omega}}_{-\upsilon} = (\tilde{\tilde{\omega}}_0, \dots, \tilde{\tilde{\omega}}_{\upsilon-1}, \tilde{\tilde{\omega}}_{\upsilon+1}, \dots, \tilde{\tilde{\omega}}_{T_l^m}), \tilde{\tilde{\omega}}^{\omega}{}_{-\upsilon} = (\tilde{\tilde{\omega}}_0, \dots, \tilde{\tilde{\omega}}_{\upsilon-1}, \omega, \tilde{\tilde{\omega}}_{\upsilon+1}, \dots, \tilde{\tilde{\omega}}_{T_l^m})$, the full conditional posterior probability of $\tilde{\tilde{\omega}}_\upsilon = \omega(\omega \in \{\tilde{\tilde{\omega}}_{\upsilon-1}, \dots, \tilde{\tilde{\omega}}_{\upsilon+1}\})$ is expressed as follows:

$$Prob\left\{\tilde{\tilde{\omega}}_\upsilon = \omega \middle| \tilde{\tilde{\omega}}_{-\upsilon}, \tilde{\tilde{d}}\right\} = \frac{\tilde{\mathcal{L}}\left(\tilde{\tilde{\omega}}^{\omega}{}_{-\upsilon}, \tilde{\tilde{d}}, \bar{\Xi}: \theta\right)}{\sum_{\omega=\tilde{\tilde{\omega}}_{\upsilon-1}}^{\tilde{\tilde{\omega}}_{\upsilon+1}} \tilde{\mathcal{L}}\left(\tilde{\tilde{\omega}}^{\omega}{}_{-\upsilon}, \tilde{\tilde{d}}, \bar{\Xi}: \theta\right)}$$

$$= \frac{\chi_\omega(\tilde{\tilde{\omega}}_{\upsilon-1}, \tilde{\tilde{\omega}}_{\upsilon+1}, \tilde{\tilde{d}})}{\sum_{\omega=\tilde{\tilde{\omega}}_{\upsilon-1}}^{\tilde{\tilde{\omega}}_{\upsilon+1}} \chi_\omega(\tilde{\tilde{\omega}}_{\upsilon-1}, \tilde{\tilde{\omega}}_{\upsilon+1}, \tilde{\tilde{d}})} \tag{3.39}$$

With the following expression:

$$\chi_\omega\left(\tilde{\tilde{\omega}}_{\upsilon-1}, \tilde{\tilde{\omega}}_{\upsilon+1}, \tilde{\tilde{d}}\right) = \omega_{\tilde{\tilde{\omega}}_{\upsilon-1}\tilde{\tilde{d}}_{\upsilon-1}}^{\omega\tilde{\tilde{d}}_\upsilon} \omega_{\omega\tilde{\tilde{d}}_\upsilon}^{\tilde{\tilde{\omega}}_{\upsilon+1}\tilde{\tilde{d}}_{\upsilon+1}} \tag{3.40}$$

In this manner, the full conditional posterior probability of the latent variables about the load bearing capacity can be determined. Due to the deterioration property of the load bearing capacity, condition (3.33b) is established unless the pavement is renewed. Here, when we assume with the latent variables that $\tilde{\tilde{d}}_{-\upsilon} = \left(\tilde{\tilde{d}}_0, \dots, \tilde{\tilde{d}}_{\upsilon-1}, \tilde{\tilde{d}}_{\upsilon+1}, \dots, \tilde{\tilde{d}}_{T_k^f}\right), \tilde{\tilde{d}}_{-\upsilon}^{d} = \left(\tilde{\tilde{d}}_0, \dots, \tilde{\tilde{d}}_{\upsilon-1}, d, \tilde{\tilde{d}}_{\upsilon+1}, \dots, \tilde{\tilde{d}}_{T_k^f}\right)$, the full conditional posterior probability of $\tilde{d}_\upsilon = d$ $(d \in \{\tilde{\tilde{d}}_{\upsilon-1}, \dots, \tilde{\tilde{d}}_{\upsilon+1}\})$ is expressed as follows:

$$Prob\left\{\tilde{\tilde{d}}_\upsilon = d \middle| \tilde{\tilde{\omega}}, \tilde{\tilde{d}}_{-\upsilon}\right\} = \frac{\chi_d(\tilde{\tilde{d}}_{\upsilon-1}, \tilde{\tilde{d}}_{\upsilon+1}, \tilde{\tilde{\omega}})}{\sum_{d=\tilde{\tilde{d}}_{\upsilon-1}}^{\tilde{\tilde{d}}_{\upsilon+1}} \chi_d(\tilde{\tilde{d}}_{\upsilon-1}, \tilde{\tilde{d}}_{\upsilon+1}, \tilde{\tilde{\omega}})} \tag{3.41}$$

With the following expression:

$$\chi_d\left(\tilde{\tilde{d}}_{\upsilon-1}, \tilde{\tilde{d}}_{\upsilon+1}, \tilde{\tilde{\omega}}\right) = \omega_{\tilde{\tilde{\omega}}_{\upsilon-1}\tilde{\tilde{d}}_{\upsilon-1}}^{\tilde{\tilde{\omega}}_\upsilon d} \omega_{\tilde{\tilde{\omega}}_\upsilon d}^{\tilde{\tilde{\omega}}_{\upsilon+1}\tilde{\tilde{d}}_{\upsilon+1}} \tag{3.42}$$

3.5 ALGORITHM

3.5.1 MCMC Method

As previously mentioned, because the likelihood function has a unique shape, the use of the normal method of maximum likelihood and Bayesian estimation is difficult for estimating the mixed distribution model including the hidden Markov deterioration model [18]. Consequently, as an estimation method for the mixed distribution model, we propose the definition of the completed likelihood function, instead of the normal likelihood function, and an estimation method for the mixed distribution model with the MCMC method [16]-[18]. To estimate the hidden Markov deterioration model, it is necessary to develop an MCMC algorithm that contains the Bayesian estimation algorithm of the Markov transition probability in the existing MCMC method for estimating the hidden Markov deterioration model. Kobayashi et al.[1] have already proposed Bayesian estimation of the hierarchic hidden Markov deterioration model with the MCMC method. Correspondingly, it is necessary to generate a series of multidimensional latent variables for the compound hidden Markov deterioration model, because the road surface soundness level and the deterioration process of the load bearing capacity experience mutual interaction.

In this study, with the MH method (Metropolis-Hastings algorithm), which is a typical MCMC method, we extract samples of unknown parameters β and γ based on the posterior probability density function. With the MH method, when sampling from posterior distribution is difficult, approximate sampling (hereafter called proposal distribution) is performed. Then, random sampling from the target distribution is performed by the inclusion of steps to correct differences between the target and proposed distribution. Here, a parameter regarding the load bearing capacity rating, s, included in load bearing capacity (3.3) and road surface deterioration model (3.9), such as $\hat{\eta}^s = (\gamma_0^s, \beta^s) = \left(\gamma_0^s, \beta_1^s, \dots, \beta_Q^s\right)$ $(s = 1, \dots, S-1)$, is an unknown parameter. As a posterior probability density function of these constant numbers, we presume a normal distribution, where the prior probability density function of parameter $\hat{\eta}^s$ is $\hat{\eta}^s \sim \mathcal{N}_{Q+1}(\zeta^{s,\hat{\eta}}, \Sigma^{s,\hat{\eta}})$. The probability density function of $Q+1$ dimensional normal distribution $\mathcal{N}_{Q+1}(\zeta^{s,\hat{\eta}}, \Sigma^{s,\hat{\eta}})$ is shown below:

$$\emptyset(\hat{\eta}^s | \zeta^{s,\hat{\eta}}, \sum^{s,\hat{\eta}}) = \frac{1}{(2\pi)^{\frac{Q+1}{2}} \sqrt{\left| \Sigma^{s,\hat{\eta}} \right|}}$$

$$exp\left\{ -\frac{1}{2}\left(\hat{\eta}^s - \zeta^{s,\hat{\eta}}\right) \left\{ \sum^{s,\hat{\eta}} \right\}^{-1} \left(\hat{\eta}^s - \zeta^{s,\hat{\eta}}\right)' \right\} \tag{3.43}$$

Here, $\zeta^{s,\hat{\eta}}$ is a prior expected value vector of $\mathcal{N}_{Q+1}(\zeta^{s,\hat{\eta}}, \Sigma^{s,\hat{\eta}})$, and $\Sigma^{s,\hat{\eta}}$ is a prior variance-covariance matrix. Similarly, we presume that a prior probability density function,

$\widehat{\mathcal{K}}^i = (\beta_0^i, \gamma_1^i, \dots, \gamma_V^i), (i = 0, \dots, I-1)$, regarding road surface soundness level i also complies with multidimensional normal distribution $\widehat{\mathcal{K}}^i$ to $\mathcal{N}_{V+1}(\zeta^{i,\mathcal{K}}, \Sigma^{i,\mathcal{K}})$. Here, $\zeta^{i,\mathcal{K}}$ is a prior expected value vector and $\Sigma^{i,\mathcal{K}}$ is a prior variance-covariance matrix.

With the random walk process Metropolis–Hastings method, we sample parameter vector $\hat{\eta}$. The random walk process Metropolis–Hastings method samples estimated parameters by random walking according to a certain probability density, and this probability density is the proposal distribution. In this study, we assume that each parameter independently complies with the random walking process according to the normal distribution, with average 0 and variance σ_e^2. In other words, concerning $e = (0, \dots, Q)$, $y = (0, \dots, V)$, we define as follows:

$$\beta_e^{(m)} - \beta_e^{(m-1)} \sim N(0, \sigma_e) \tag{3.44a}$$

$$\gamma_y^{(m)} - \gamma_y^{(m-1)} \sim N(0, \sigma_y) \tag{3.44b}$$

The letter m indicates the number of sampling times. In the next section, we create the sampling procedure of parameter vectors $\hat{\eta}$ and $\widehat{\mathcal{K}}$ with the random walk process MH method.

a) Step 1: Establishing initial value

We establish variance parameters σ_e and σ_y of proposal distributions (3.44a) and (3.44b) at random. We establish initial virtual latent variables $\tilde{\tilde{d}}^1 = (\tilde{\tilde{d}}_0^{(0)}, \dots, \tilde{\tilde{d}}_{u_k^f}^{(0)}, \dots, \tilde{\tilde{d}}_T^{(0)})$ and $\tilde{\tilde{\omega}}^{(0)} = (\tilde{\tilde{\omega}}_0^{(0)}, \dots, \tilde{\tilde{\omega}}_{u_l^m}^{(0)}, \dots, \tilde{\tilde{\omega}}_T^{(0)})$. These must meet restriction conditions (3.33a) and (3.33b).

Additionally, we establish estimated parameter values $\hat{\eta}^{(0)}$ and $\widehat{\mathcal{K}}^{(0)}$ at random. The influence of these initial values is weakened by the number of simulations accumulated with the MCMC method. The number of times of sampling, m, is defined as $m = 1$.

b) Step 2: Sample extraction of parameter $\hat{\eta}^{(m)}$

We extract a sample of the deterioration hazard model, defined concerning the load bearing capacity rank, s, of pavement $\hat{\eta}^{s,(m)} = (\hat{\gamma}_0^{s,(m)}, \beta_1^{s,(m)}, \dots, \beta_Q^{s,(m)})$, $(s = 1, \dots, S-1)$ with the random walk process MH method.

Step 2-1: The virtual latent variable vectors, $\tilde{\tilde{\omega}}^{(m-1)}$ and $\tilde{\tilde{d}}^{(m-1)}$, and the parameter vectors, $\hat{\eta}^{(m-1)}$ and $\widehat{\mathcal{K}}^{(m-1)}$, are given conditions.

Step 2-2: We define the parameter vector of which the number of sampling times is m and the substep is g, as follows:

$$\hat{\eta}_{g-1}^{s,(m)} = (\hat{\gamma}_0^{s,(m)}, \dots, \beta_{g-1}^{s,(m)}, \beta_g^{s,(m-1)}, \dots, \beta_Q^{s,(m-1)})' \tag{3.45}$$

Additionally, we define $\iota_g^{(m)} = (0, \dots, 0, \iota_g^{s,(m)}, 0, \dots, 0)'$, which is a random walk vector of substep g (a column vector of which element g only takes the value $\iota_g^{s,(m)}$). Based on the assumption that a step width of the random walking process complies with the normal distribution, with average 0 and variance $(\sigma)^2$, the following relation is established:

$$\iota_g^{s,(m)} \sim N(0, (\sigma_g)^2)$$

Here, we define parameter vector $\hat{\eta}_g^{s,(m)}$ as follows:

$$\hat{\eta}_g^{s,(m)} = \hat{\eta}_{g-1}^{s,(m)} + \iota_g^{s,(m)} \tag{3.46}$$

and parameter vector $\hat{\eta}_{(s,g)}^{(m)}$ as follows:

$$\hat{\eta}_{(s,g)}^{(m)} = \left(\hat{\eta}^{1,(m)}, \dots, \hat{\eta}_g^{s,(m)}, \hat{\eta}^{s+1,(m-1)}, \dots, \hat{\eta}^{S-1,(m-1)}\right) \tag{3.47}$$

Next, we define likelihood ratio $\Upsilon_{(s,g)}^{(m)}$ as follows:

$$\Upsilon_{(s,g)}^{(m)} = min\left[\frac{\tilde{\mathcal{L}}\left(\beta_{(s,g)}^{(m)}, \bar{\Xi}\right)}{\tilde{\mathcal{L}}\left(\beta_{(s,g-1)}^{(m)}, \bar{\Xi}\right)}, 1\right]$$

Here, $\tilde{\mathcal{L}}\left(\beta_{(s,g)}^{(m)}, \bar{\Xi}\right) = \tilde{\mathcal{L}}\left(\tilde{\tilde{\omega}}^{(m-1)}, \tilde{d}^{(m-1)}, \beta_{(s,g)}^{(m)}, \tilde{\tilde{\gamma}}^{(m-1)}, \bar{\Xi}\right)$ is the completed likelihood function shown in formula (3.38).

Step 2-3: Based on the uniform distribution, $U(0,1)$, defined in section [0, 1], we generate uniform distribution u to $U(0,1)$, and determine $\beta^{m,g}$ according to the following rules:

$$\beta_g^{s,(m)} = \begin{cases} \beta_{g-1}^{s,(m)} + \iota_g^{s,(m)} & in\ the\ case\ u \le \Upsilon_{(s,g)}^{m} \\ \beta_{g-1}^{s,(m)} & Other\ cases \end{cases} \tag{3.48}$$

We operate this procedure from $g = 0$ to $g = Q$.

c) Step 3: Sample extraction of parameter $\gamma^{(m)}$

We extract a sample of the parameter vector $\gamma^{(m-1)} = (\gamma_1^{(m-1)}, \dots, \gamma_G^{(m-1)})$ of the deterioration hazard model concerning the road surface soundness level with the random walk process MH method.

Step 3-1: Virtual latent variable vectors $\tilde{\tilde{\omega}}^{(m-1)}$ and $\tilde{\tilde{d}}^{(m-1)}$, and parameter vectors $\hat{\beta}^{(m)}$ and $\hat{\gamma}^{(m-1)}$, are given conditions.

Step 3-2: We define the parameter vectors of which the number of sampling times is m and the substep is y, as follows:

$$\gamma_{y-1}^{i,(m)} = \left(\gamma_1^{i,(m)}, \dots., \gamma_{y-1}^{i,(m)}, \gamma_y^{i,(m)}, \dots, \gamma_V^{i,(m)}\right)'$$

Additionally, we define $\iota_y^{i,(m)} = (0, \dots, 0, \iota_y^{i,(m)}, 0, \dots, 0)'$, which is a random walk vector of substep y (a column vector of which i element only takes the value from $\iota_y^{i,(m)}$ to $N\left(0, (\sigma_y)^2\right)$. Next, we determine the likelihood ratio as follows:

$$\Upsilon_{i,y}^{(m)} = min\left[\frac{\tilde{\mathcal{L}}\left(\gamma_y^{i,(m)}, \bar{\Xi}\right)}{\tilde{\mathcal{L}}\left(\gamma_{y-1}^{i,(m)}, \bar{\Xi}\right)}, 1\right]$$

Step 3-3: Based on uniform distribution $U(0,1)$ defined in section [0, 1], we generate uniform distribution u to $U(0,1)$, and determine $\gamma_y^{i,(m)}$ according to the following rules:

$$\gamma_y^{i,(m)} = \begin{cases} \gamma_{y-1}^{i,(m)} + \iota_y^{i,(m)} & in\ the\ case\ u \le \Upsilon_{(i,y)}^{(m)} \\ \gamma_{y-1}^{i,(m)} & Other\ cases \end{cases} \tag{3.49}$$

We operate this procedure from $y = 0$ to $y = V$.

d) Step 4: Sample extraction of latent variable $\tilde{\omega}^{(m)}$

With given conditions such as virtual latent variable vectors $\tilde{\tilde{\omega}}^{(m-1)}$ and $\tilde{\tilde{d}}^{(m-1)}$, and parameter vectors $\hat{\beta}^{(m)}$ and $\hat{\gamma}^{(m)}$, we extract a new virtual latent variable sample, $\tilde{\tilde{\omega}}^{(m)}$. With given conditions such as updated parameter estimations $\hat{\beta}^{(m)}$ and $\hat{\gamma}^{(m)}$, Markov transition $\omega_{is}^{jv}(m)$ with formula (3.15) is defined. Based on full conditional posterior probability (3.39), we randomly sample a new virtual latent variable, $\tilde{\tilde{\omega}}^{(m)}$. Let us focus on a certain period, $[t_l^m,\ t_{l+1}^m)$ $(l = 0, \dots, N^m - 1)$, and define latent variable vector $\tilde{\tilde{\omega}}_{-v}^{(m)} = \left(\tilde{\tilde{\omega}}_1^{(m)}, \dots, \tilde{\tilde{\omega}}_{v-1}^{(m)}, \tilde{\tilde{\omega}}_{v+1}^{(m-1)}, \dots, \tilde{\tilde{\omega}}_{T_l^m}^{(m-1)}\right)$. At this time, the full conditional posterior probability of $\tilde{\tilde{\omega}}_v^{(m)} = \omega \in \left\{\tilde{\tilde{\omega}}_{v-1}^{(m)}, \dots, \tilde{\tilde{\omega}}_{v+1}^{(m-1)}\right\}$ is expressed as follows:

$$Prob\left\{\tilde{\tilde{\omega}}_v = \omega \middle| \tilde{\tilde{\omega}}_{-v}^{(m)}, \tilde{\tilde{d}}^{(m-1)}\right\} = \frac{\chi_\omega\left(\tilde{\tilde{\omega}}_{v-1}^{(m)}, \tilde{\tilde{\omega}}_{v+1}^{(m-1)}, \tilde{\tilde{d}}^{(m-1)}\right)}{\sum_{\omega=\tilde{\tilde{\omega}}_{v-1}^{(m)}}^{\tilde{\tilde{\omega}}_{v+1}^{(m-1)}} \chi_\omega\left(\tilde{\tilde{\omega}}_{v-1}^{(m)}, \tilde{\tilde{\omega}}_{v+1}^{(m-1)}, \tilde{\tilde{d}}^{(m-1)}\right)} \tag{3.50}$$

With the formula below:

$$\chi_\omega\left(\tilde{\tilde{\omega}}_{v-1}^{(m)}, \tilde{\tilde{\omega}}_{v+1}^{(m-1)}, \tilde{\tilde{d}}^{(m-1)}\right) = \omega_{\tilde{\tilde{\omega}}_{v-1}^{(m)}\tilde{\tilde{d}}_{v-1}^{(m-1)}}^{\omega\tilde{\tilde{d}}_v^{(m-1)}}(m)\omega_{\omega\tilde{\tilde{d}}_v^{(m-1)}}^{\tilde{\tilde{\omega}}_{v+1}^{(m-1)}\tilde{\tilde{d}}_{v+1}^{(m-1)}}(m) \tag{3.51}$$

We sequentially find latent variable $\tilde{\tilde{\omega}}_v^{(m)}(v = 0, \dots, T_l^m)$ for all l $(l = 0, \dots, N^m - 1)$ based on $v = 0$.

e) Step 5: Sample extraction of latent variable $\tilde{d}^{(m)}$

With given conditions such as virtual latent variable vectors $\tilde{\tilde{\omega}}^{(m)}$ and $\tilde{\tilde{d}}^{(m-1)}$, and parameter vectors $\hat{\beta}^{(m)}$ and $\hat{\gamma}^{(m)}$, we extract a new virtual latent variable sample, $\tilde{\tilde{d}}^{(m)}$. Let us focus on a certain period, $[t_k^f,\ t_{k+1}^f)$ $(k = 0, \dots, N^f - 1)$, and define latent variable vector $\tilde{\tilde{d}}_{-\upsilon}^{(m)} = \left(\tilde{\tilde{d}}_1^{(m)}, \dots, \tilde{\tilde{d}}_{\upsilon-1}^{(m)}, \tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}, \dots, \tilde{\tilde{d}}_{T_k^f}^{(m-1)}\right)$. At this time, the full conditional posterior probability of $\tilde{\tilde{d}}_\upsilon^{(m)} = d \in \left\{\tilde{\tilde{d}}_{\upsilon-1}^{(m)}, \dots, \tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}\right\}$ is expressed as follows:

$$Prob\left\{\tilde{\tilde{d}}_\upsilon = d \,\middle|\, \tilde{\tilde{\omega}}^{(m)}, \tilde{\tilde{d}}_{-\upsilon}^{(m)}\right\} = \frac{\chi_d\left(\tilde{\tilde{\omega}}^{(m)}, \tilde{\tilde{d}}_{\upsilon-1}^{(m)}, \tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}\right)}{\sum_{d=\tilde{\tilde{d}}_{\upsilon-1}^{(m)}}^{\tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}} \chi_d\left(\tilde{\tilde{\omega}}^{(m)}, \tilde{\tilde{d}}_{\upsilon-1}^{(m)}, \tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}\right)} \tag{3.52}$$

With the formula below:

$$\chi_d\left(\tilde{\tilde{\omega}}^{(m)}, \tilde{\tilde{d}}_{\upsilon-1}^{(m)}, \tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}\right) = \omega_{\tilde{\tilde{\omega}}_{\upsilon-1}^{(m)}\tilde{\tilde{d}}_{\upsilon-1}^{(m-1)}}^{\tilde{\tilde{\omega}}_\upsilon^{(m)}d}(m)\,\omega_{\tilde{\tilde{d}}_\upsilon^{(m)}d}^{\tilde{\tilde{\omega}}_{\upsilon+1}^{(m)}\tilde{\tilde{d}}_{\upsilon+1}^{(m-1)}}(m) \tag{3.53}$$

We sequentially find latent variable $\tilde{\tilde{d}}_\upsilon^{(m)}$ $(\upsilon = 0, \dots, T_k^f)$ for all k $(k = 0, \dots, N^f - 1)$ based on $\upsilon = 0$.

f) Step 6: Decision on algorithm completion

We record updated parameter estimation values $\beta^{(m)}$ and $\gamma^{(m)}$, and updated latent variables $\tilde{\tilde{\omega}}^{(m)}$ and $\tilde{\tilde{d}}^{(m-1)}$ discussed in the previous section. In the case of $m \le \overline{m}$, considering $m = m + 1$, we return to step 2. Otherwise, we terminate the algorithm.

The initial step of the preceding algorithm remains affected by the initial parameter values. Therefore, it is recommended to remove generated parameter samples on the assumption that the parameter sample generation process does not reach the normal process until the number of simulations (m) becomes sufficiently large. Here, we express the minimum number of simulations (m) to be adopted as a parameter sample, $\underline{m}$. In this case, with the samples found by Gibbs sampling such as $\beta^{(m)}$ and $\gamma^{(m)}$ $(m = \underline{m} + 1, \underline{m} + 2, \dots, \overline{m})$, we can calculate various types of statistics regarding the posterior distribution of parameter vectors β and γ.

3.5.2 Statistics Regarding Posterior Distribution

Based on the samples acquired with the MCMC method, we can analyze the statistical characteristics of parameter vectors β and γ. With the MCMC method, we cannot express the posterior probability density function of the parameters as an analytical function; thus, it is necessary to estimate distribution and density functions nonparametrically with the acquired samples. Here, we express the samples acquired with the MCMC method as $\theta^{(m)} = \left(\beta^{(m)}, \gamma^{(m)}\right)$ $(m = 1, \dots, \overline{m})$. Among these, $\underline{m}$ samples are first considered to be samples from the convergence process and removed from the group of samples. Then, we define a

group of sample subscripts with parameters as $\mathcal{M} = \{\underline{m}+1, \dots, \overline{m}\}$. Because the statistics of parameters β and γ can be defined in the same way, we focus on parameter β for discussion below. First, a joint probability distribution function, $G(\beta)$, of parameter β can be expressed as follows:

$$G(\beta) = \frac{\#\left(\beta^{(m)} \leq \beta \ , m \in \mathcal{M}\right)}{\overline{m} - \underline{m}} \tag{3.54}$$

Here, $\#\left(\beta^{(m)} \leq \beta \ , m \in \mathcal{M}\right)$ is the total number of samples in which the logical formula $\beta^{(m)} \leq \beta \ , m \in \mathcal{M}$ is established. Additionally, the expected value vector, $\tilde{\zeta}^s(\beta^s)$, and variance-covariance matrix, $\widetilde{\sum^s}\ (\beta^s)$, of the posterior distribution of parameter β^s are individually expressed as follows:

$$\tilde{\zeta}^s(\beta^s) = \left(\tilde{\zeta}^s(\beta_0^s), \dots, \tilde{\zeta}^s\left(\beta_Q^f\right)\right)' = \left(\sum_{m=\underline{m}+1}^{\overline{m}} \frac{\beta_0^{s(m)}}{\overline{m} - \underline{m}}, \dots, \sum_{m=\underline{m}+1}^{\overline{m}} \frac{\beta_Q^{s(m)}}{\overline{m} - \underline{m}}\right)' \tag{3.55a}$$

$$\widetilde{\sum^s}\ (\beta^s) = \begin{pmatrix} \tilde{\sigma}^2(\beta_0^s) & \cdots & \tilde{\sigma}\left(\beta_0^s \beta_Q^s\right) \\ \vdots & \ddots & \vdots \\ \tilde{\sigma}\left(\beta_Q^s \beta_0^s\right) & \cdots & \tilde{\sigma}^2\left(\beta_Q^s\right) \end{pmatrix} \tag{3.55b}$$

The following formulas correspond to $r, q = 0, \cdots, Q$:

$$\tilde{\sigma}^2(\beta_r^s) = \sum_{m=\underline{m}+1}^{\overline{m}} \frac{\left\{\beta_r^{s(m)} - \tilde{\zeta}(\beta_r^s)\right\}^2}{\overline{m} - \underline{m}} \tag{3.56a}$$

$$\tilde{\sigma}\left(\beta_r^s \beta_q^s\right) = \sum_{m=\underline{m}+1}^{\overline{m}} \frac{\left\{\beta_r^{s(m)} - \tilde{\zeta}(\beta_r^s)\right\}\left\{\beta_q^{s(m)} - \tilde{\zeta}\left(\beta_q^s\right)\right\}}{\overline{m} - \underline{m}} \tag{3.56b}$$

Additionally, we can define a confidence interval of parameter β with the parameter samples acquired by using the MCMC method. For example, confidence interval $100(1-2\varepsilon)\%$ of parameter β can be expressed as $\beta_{-q}^{s,\varepsilon} < \beta_q^s < \beta_q^{-s,\varepsilon}$ with the following sample order $(\beta_{-q}^{s,\varepsilon}, \beta_q^{-s,\varepsilon})$ $(s = 1, \dots, S-1; q = 0, \dots, Q)$ statistics:

$$\beta_{-q}^{s,\varepsilon} = \arg\max_{\beta_q^{s*}} \left\{\frac{\#\left(\beta_q^{s(m)} \leq \beta_q^{s*} \quad \in \mathcal{M}\right)}{\overline{m} - \underline{m}} \leq \varepsilon\right\} \tag{3.57a}$$

$$\overline{\beta}_q^{s,\varepsilon} = \arg\min_{\beta_q^{s**}} \left\{ \frac{\#\left(\beta_q^{s(m)} \geq \beta_q^{s**} \quad \in \mathcal{M}\right)}{\overline{m} - \underline{m}} \leq \varepsilon \right\} \tag{3.57b}$$

When using the MCMC method, there is no guarantee that the initial parameter value, $\theta^{(0)}$, is a sample from posterior distribution, which is an invariant distribution. Among $\overline{m}$ samples generated in Gibbs sampling, we consider that the first $\underline{m}$ samples, $\theta^{(m)} = \left(\beta^{(m)}, \gamma^{(m)}\right)$ $(m = 1, \ldots, \underline{m})$, arise from sampling during the convergence process to posterior distribution. Then, we use the samples at $\underline{m} + 1$ time and later as examples. We attempt to test of hypothesis to confirm whether these samples are from posterior distribution, which is an invariant distribution, with the Geweke method [19]. Here, from the Gibbs sample, $\theta^{(m)}$ $(m = 1, \ldots, \overline{m})$, of the parameter, let us take the first m_1 data and last m_2 data. Geweke recommended $m_1 = 0.1\left(\overline{m} - \underline{m}\right), m_2 = 0.5\left(\overline{m} - \underline{m}\right)$ [19]. At this time, the Geweke test statistic to judge convergence of parameter β to invariant distribution can be defined as follows:

$$Z_{\beta_q^s} = \frac{{}_1\overline{\beta}_q^s \quad - \quad {}_2\overline{\beta}_q^s}{\sqrt{v_1^2\left(\beta_q^s\right) + v_2^2\left(\beta_q^s\right)}} \sim \mathcal{N}(0,1) \tag{3.58}$$

$${}_1\overline{\beta}_q^s = \sum_{m=\underline{m}+1}^{\overline{m}+m_1} \beta_q^{s(m)} \qquad {}_2\overline{\beta}_q^s = \frac{\sum_{m=\overline{m}-m_2+1}^{\overline{m}} \beta_q^{s(m)}}{m_2}$$

$$v_1^2\left(\beta_q^s\right) = \frac{2\pi \hat{f}_{\beta_q^s}^1(0)}{m_1} \qquad v_2^2\left(\beta_q^s\right) = \frac{2\pi \hat{f}_{\beta_q^s}^2(0)}{m_2}$$

Here, $f_{\beta_q^s}^l(x)$ $(l = 1,2)$ is a spectral density function, and an estimated value of $2\pi f_{\beta_q^s}^l(0)$ can be found as follows [20]-[22]:

$$2\pi \hat{f}_{\beta_q^s}^l(0) = \quad {}_l\widehat{\omega}_0 + 2\sum_{s=1}^{q} \omega(s,q) \quad {}_l\widehat{\omega}_q^s \tag{3.59}$$

$${}_1\widehat{\omega}_0 = m_1^{-1} \sum_{m=\underline{m}+1}^{\underline{m}+m_1} \left(\beta_q^{s(m)} \quad - \quad {}_1\overline{\beta}_q^s\right)^2$$

$${}_2\widehat{\omega}_0 = m_2^{-1} \sum_{m=\overline{m}-m_2+1}^{\overline{m}} \left(\beta_q^{s(m)} \quad - \quad {}_2\overline{\beta}_q^s\right)^2$$

$$ {}_1\widehat{\omega}_q^s = m_1^{-1} \sum_{m=\underline{m}+s+1}^{\underline{m}+m_1} \left(\beta_q^{s(m)} - {}_1\overline{\beta}_q^s\right)\left(\beta_q^{s(m-s)} - {}_1\overline{\beta}_q^s\right) $$

$$ {}_2\widehat{\omega}_q^s = m_2^{-1} \sum_{m=\overline{m}-m_2+s+1}^{\overline{m}} \left(\beta_q^{s(m)} - {}_2\overline{\beta}_q^s\right)\left(\beta_q^{s(m-s)} - {}_2\overline{\beta}_q^s\right) $$

$$ \omega(s,q) = 1 - \frac{f}{v+1} $$

v is the parameter showing the approximate value of the spectral density, and we adopt 3.20 for this parameter according to Geweke [19]. Here, we establish a null hypothesis, H_0, and an alternative hypothesis, H_1, of convergence properties of β_q^s in invariant distribution as follows:

$$ \begin{cases} H_0: \left|Z_{\beta_q^s}\right| \leq Z_{\upsilon/2} \\ H_1: \left|Z_{\beta_q^s}\right| > Z_{\upsilon/2} \end{cases} \tag{3.60} $$

Here, $Z_{\upsilon/2}$ is a critical value to reject the null hypothesis. When testing the null hypothesis based on the level of significance, υ, $Z_{\upsilon/2}$ can be defined as the value that satisfies $\upsilon/2\% = 1 - \Phi(Z\_(\upsilon/2);$ Φ (Z) is the distribution function of standard normal distribution.

3.6 APPLICATION EXAMPLE

3.6.1 Database Outline

Using the pavement survey data collected for the expressways managed by NEXCO, we discuss the effectiveness of the method proposed in this study. The data used in this study were collected by the NEXCO Research Institute (NEXCO RI), which randomly selected road sections throughout the country and conducted FWD and road condition surveys on them for eight years, from April 2006 to March 2014. In total, road condition surveys were conducted at 1,726 road sections; NEXCO RI conducted road condition surveys at 7,568 locations, and FWD surveys were carried out at 39,663 locations. The road sections described here were investigated at multiple time cross sections. Additionally, during the target period, although road surface repairs were conducted, the pavements were not updated. Because data were available at the service start time, to collect sample data, we individually defined a period from the service start to the initial survey, and another period from the current survey to the next survey as one unit. In other words, when a survey was conducted for the road section *n* times, we collected sample data in *n* units. Based on this concept, database to estimate the compound

hidden Markov deterioration model was prepared. For model estimation, 5,269 samples were used for road surface soundness level and 6,103 for load bearing capacity.

Table 3.1 Sample data features

Total length	3,400.00 km						
Year of construction	Vary from 1964 to 2014						
Total number of road sections	1,726						
Total number of soundness level samples	5,269						
Total number of load bearing capacity samples	6,103						
Traffic volume of large vehicles	3,930 vehicles on average (103 to 13,765)						
Sampling condition by load bearing capacity	Load bearing capacity	1	2	3	4	5	
	Number of samples	1,387	1,713	1,583	790	630	
	Rate of samples	23%	28%	26%	13%	10%	
Sampling condition by road surface soundness level	Soundness level	1	2	3	4	5	6
	Number of samples	3,025	1,403	352	218	115	156
	Rate of samples	57%	27%	7%	4%	2%	3%

Table 3.2 Definition of road surface soundness level

Soundness level	Cracking ratio (%)
1	$Cr = 0$
2	$0 < Cr \leq 2.5$
3	$2.5 < Cr \leq 5$
4	$5 < Cr \leq 10$
5	$10 < Cr \leq 20$
6	$20 < Cr$

Table 3.1 shows features of the sample data prepared as mentioned above. In a road condition survey, we evaluate the road surface soundness level in basic units of 100-m sections, and select one part to be repaired according to the evaluation result. In a road condition survey, we can gather individual information about road surface soundness levels concerning three damage types: (1) crack, (2) rutting, and (3) IRI. The standard values for repair have been set in advance for these three types of damage, and when even one of these types in a certain pavement reaches the standard value, it is detected as a section that needs repair. In this study, among the three damage types, we focus on pavement crack in cracking ratio, which is

considered to proceed rapidly, and define the road surface soundness levels. Table 3.2 lists definitions of the road surface soundness rankings based on range of cracking ratio. The ranking 6 describes the limit of pavement surface conditions for operation with cracking ratio of 20 percent. Once pavement cracking ratio reaches such ranking, repair must be requested. Additionally, in the same target road sections where the road surface surveys are conducted, deflection measurement with FWD is implemented. In this section, we define the soundness levels of the pavement load bearing capacity with a damage index acquired by using the measured deflection data. NEXCO RI establishes the damage index shown below to evaluate damage of asphalt layers (hereafter called "As layers") [23],[24].

$$D_i = \frac{D_0 - D_{90}}{\Delta} \tag{3.61}$$

Here, D_0 indicates deflection (mm) directly under the center of falling-mass loading, D_{90} indicates deflection (mm) at a point 90 cm away from the center of falling-mass loading, and Δ indicates the design thickness (mm) of the As layers. In this section, we discuss the use of the previously mentioned method. Damaged segments of As layers are grouped into A, B, and C, according to the damage index, prepared corresponding to combinations of damage by top surface type (highly functional pavement I, highly functional pavement II, and dense-graded pavement), subgrade type (granular-base subgrade and cement stabilizing treatment subgrade), and As layers design thickness (less than 220 mm, 220mm to 260 mm, and 260 mm or more). Table 3.3 lists the individually estimated damage layers. Because we could not collect sufficient samples of highly functional pavement II or dense-graded pavement, we used limited samples of highly functional pavement I, which is the pavement type adopted by NEXCO as a standard. On the assumption that soundness level 5 is the application limit of the load bearing capacity, we established values that were double the thresholds of damage segments A and B. Additionally, we divided the thresholds of damage segments A and B into quarters and corresponded them to each soundness level from 1 to 4. Consequently, Table 3.4 defines the soundness levels based on the load bearing capacity by subgrade type and design thickness of As layers. Because values over the application limits (in other words, double the thresholds of damage segments A and B) supposedly indicated a decrease in the load bearing capacity of not only the As layers, but also subbase, roadbed, and soil, we considered them improper as samples and deleted them. It is generally difficult to find a relation between a real evaluation of the cracking ratio, of which the evaluation length is 100 m, and point evaluation, in which only one point on the outer wheel path in the same 100m-length section is evaluated, such as in FWD surveys. In other words, similar processes are used to find a relation between an area and a point, which are completely different objects. Actually, we decided to compare an FWD survey point with a newly defined cracking ratio for a small area that includes the particular survey point. However, in fact, NEXCO strictly selects a point at which the road surface is damaged worst in a 100-m section and conducts a conventional FWD survey for that point. We will develop such newly defined cracking ratios in the future, but for this study, we used the

FWD survey results as typical load bearing capacity for the 100m evaluation length and operation estimations, as is the current custom with NEXCO.

Table 3.3 Damage segment and estimated damaged layer

Damage segment	Estimated damaged layer
A	All layers of As layers (top surface, base, and upper subgrade)
B	Top surface and base of As layers
C	Although the top surface of As layers is supposedly damaged, the overall structure is sound

Table 3.4 Definition of load bearing capacity $\boldsymbol{Di}$ ($\boldsymbol{mm}$)

Damage level	By top surface type	Highly functional pavement I					
	By subgrade type	Granular-base subgrade			Cement stabilizing treatment subgrade		
	As layers thickness	≤ 220	(220, 260)	≥ 260	≤ 220 mm	(220, 260)	≥ 260
1		[0, 375]	[0, 200]	[0, 125]	[0, 300]	[0, 200]	[0, 100]
2		(375, 750]	(200, 400]	(125, 250]	(300, 600]	(200, 400]	(100, 200]
3		(750, 1125]	(400, 600]	(250, 375]	(600, 900]	(400, 600]	(200, 300]
4		(1125, 1500]	(600, 800]	(375, 500]	(900, 1200]	(600, 800]	(300, 400]
5		(1500, 3000]	(800, 1600]	(500, 1000]	(1200, 2400]	(800, 1600]	(400, 800]

Table 3.5 Large vehicle traffic segment

(unit: one vehicle/day/one direction)

Section classification in term of traffic condition	Details
Light traffic section	Less than 1500
Medium traffic section	1,500 to less than 5,000
Heavy traffic section	5,000 or more

3.6.2 Results of Model Estimation

We estimated load bearing capacity hazard model (3.3) for four load bearing capacity levels in total, except for load bearing capacity level 5, listed in Table 3.4. Additionally, as shown in formula (3.3), the load bearing capacity hazard rate includes the parameter β_0^i $(i = 1, \dots, I - 1)$, which shows that the deterioration hazard rate changes proportionally according to the road surface soundness level. Among $I - 1$ parameters β_0^i, we standardized β_0^1 as 1, based on the parameter identification possibility condition. Additionally, we estimated road surface soundness level hazard rates (3.9) for five soundness levels in total, except for soundness level 6, listed in Table 3.2. As shown in formula (3.9), concerning the road surface soundness level hazard rates, parameter γ_0^s $(s = 1, \dots, S - 1)$ is included, which shows that the deterioration hazard rate changes proportionally according to the load bearing capacity level. Among $S - 1$

parameters γ_0^s, we standardized γ_0^1 as 1. For the case in which formula $\beta_0^i = 1$ is established for approximately every $(i = 2, \dots, I-1)$ data, or formula $\gamma_0^s = 1$ is established for approximately every s $(s = 2, \dots, S-1)$, the hazard rates of the road surface soundness level and the load bearing capacity are independent, and the compound deterioration hypothesis, which states that the deterioration of the road surface soundness level and load bearing capacity affect each other, can be rejected. We will discuss such a statistical test problem of the compound deterioration hypothesis in the following section [3].

Table 3.6 Parameter estimation result (load bearing capacity)

Load bearing capacity: s	Constant term: β_1^s	Regional feature: General = 1 Cold area = 0; β_2^s	Large vehicle traffic segment: Heavy = 1 Light and medium = 0; β_3^s	Hazard rate $E\,[\lambda^S(1)]$	Expected life $ET^S(1)$ (year)
1	-2.170 (-2.295, -2.071) -0.036	– (–) –	0.850 (0.668, 0.923) 0.135	0.114	8.75
2	-1.748 (-1.632, -1.887) -0.022	-0.411 (-0.672, -0.343) 0.013	– (–) –	0.115	8.67
3	-1.819 (-1.917, -1.764) -0.008	– (–) –	1.469 (1.315, 1.622) 0.264	0.162	6.17
4	-1.116 (-1.224, -0.982) 0.019	– (–) –	– (–) –	0.328	3.05

Note: For each soundness level, the first line shows expected value of the parameter, the second line shows lower and upper limits of the 95% confidence interval of the estimated parameter values, and the third line shows the Geweke test statistic.

Table 3.7 Parameter estimation result (road surface soundness level)

Road surface soundness level: i	Constant term: γ_1^i	Large vehicle traffic segment: Heavy = 1 Light and medium = 0; γ_2^i	Hazard rate: $E[\mu^i(1)]$	Expected life: $ET^i(1)$ (year)
1	-2.375 (-2.462, -2.254) 0.037	1.456 (1.228, 1.575) -0.002	0.093	10.75
2	-1.232 (-1.015, -1.473) 0.019	– (–) –	0.292	3.43
3	0.693 (0.452, 0.800) 0.036	– (–) –	1.999	0.50
4	1.338 (1.315, 1.622) -0.072	– (–) –	3.810	0.26
5	1.844 (1.722, 1.905) -0.019	2.134 (2.027, 2.336) -0.193	6.319	0.16

Note) For each soundness level, the first line shows expected value of the parameter, the second line shows lower and upper limits of the 95% confidence interval of the estimated parameter values, and the third line shows the Geweke test statistic. $E[\mu^i(1)]$ *shows expected hazard rate of road surface soundness level* i *in the case of* $s = 1$, *and* $ET^i(1)$ *(year) shows expected life of road surface soundness level i in the case of* $s = 1$.

Table 3.8 Scale parameter estimation result (compound model β^0)

Road surface soundness level: i	Scale parameter: β_0^i
i = 2	1.697 (1.692, 2.031) -0.098
i = 3	2.788 (2.492, 3.361) 0.137
i = 4	3.200 (3.063, 3.493) -0.194
i = 5	3.568 (3.296, 3.722) -0.107

Note) For each soundness level, the first line shows expected value of the parameter, the second line shows lower and upper limits of the 95% confidence interval of the estimated parameter values, and the third line shows the Geweke test statistic.

Table 3.9 Scale parameter estimation result (compound model γ^0)

Load bearing capacity: s	Scale parameter: γ_0^s
s = 2	2.412 (2.115, 2.653) 0.089
s = 3	3.270 (3.042, 3.414) 0.038
s = 4	4.008 (3.633, 4.221) -0.017

Note) For each soundness level, the first line shows expected value of the parameter, the second line shows lower and upper limits of the 95% confidence interval of the estimated parameter values, and the third line shows the Geweke test statistic.

Potential factors affecting the hazard rates of the road surface soundness level and the load bearing capacity are regional features, pavement thickness, As layer thickness, road structure features, and large vehicle traffic volumes. We estimated a compound hidden Markov pavement deterioration hazard function for each combination of these candidate explanatory variables. We standardized explanatory variables that could be measured quantitatively, such as pavement thickness, so that the maximum value in the samples was 1. Additionally, for the As layer thickness, we established dummy variables 1 or 0 according to the design thickness of the As layer (less than 220 mm, 220m to 260 mm, and 260 mm or more), which was the definition of the load bearing capacity level, so that the dummy variables or a combination of the variables allowed distinction. Furthermore, we divided the large vehicle traffic volume into three sections using the traffic volume database of 2011, based on the pavement traffic volume sections in the technical standard ("Design Guideline No. 1: Pavement") of NEXCO. Table 3.5 lists these traffic sections. In the combinations of explanatory variables, we excluded combinations of the variables that did not satisfy the sign conditions and Geweke test, and finally determined the optimal combination of these variables in terms of Akaike Information Criterion (AIC) engineering. For the previously discussed compound hidden Markov hazard

models we estimated, Table 3.6 and Table 3.7 list the estimation results of the hazard models indicating the deterioration processes of the load bearing capacity and the road surface, respectively. As explanatory variables, regional features and large vehicle traffic volume sections were adopted for load bearing capacity, and large vehicle traffic volume sections were adopted for road surface soundness level. Table 3.8 lists the scale parameter estimation results indicating the heterogeneity of deterioration speed of the load bearing capacity, which is dependent on the road surface soundness conditions; Table 3.9 lists the scale parameter estimation results indicating the heterogeneity of road surface deterioration speed, which is dependent on the load bearing capacity. For example, when road surface soundness level i is 2, scale parameter β_0^2 is 1.697, which indicates that the deterioration speed of the load bearing capacity is 1.697 times faster than in the case when i is 1. We established the number of samples, $\underline{m}$, at 2,000 to make the Markov chain reach static state in Gibbs sampling with the MCMC method. As listed in Table 3.6 to Table 3.9, each Geweke test statistic is under 1.96, and the convergence hypothesis, which means that the random sampling of parameters converges to static state under a 5% level of significance, cannot be rejected. Therefore, in this study, we set $\overline{m}$ at $12{,}000$ and excluded $\underline{m} = 2{,}000$ samples from the convergence process to posterior distribution, and used the remaining $10{,}000$ parameter samples for analysis.

With the load bearing capacity hazard rate, $\lambda^s(i)$, and road surface soundness level hazard rate, $\mu^i(s)$, in a relevant section, an expected life, $ET^s(i)$ (required time until the deterioration condition worsens), of the load bearing capacity, s, for a case in which the road surface soundness level is i, and the expected life, $ET^i(s)$, of i for a case in which the load bearing capacity is s, are defined individually as follows:

$$ET^s(i) = \int_0^\infty exp(-\lambda^s(i)y^s)dy^s = \frac{1}{\lambda^s(i)} \tag{3.62a}$$

$$ET^i(s) = \frac{1}{\mu^i(s)} \tag{3.62b}$$

Table 3.6 lists the expected hazard rate and life of s for a case in which i is 1, for each load bearing capacity level. Similarly, Table 3.7 lists the expected hazard rate and life of i for a case in which s is 1, for each road surface soundness level. Additionally, Table 3.10 lists each expected hazard rate and expected life by load bearing capacity, found for each i. According to this table, the expected life of the load bearing capacity shortens as the road surface soundness level deteriorates. Similarly, Table 3.11 lists each expected hazard rate and expected life by road surface soundness level, found for each s. According to this table, the expected life of the road surface soundness level shortens as the load bearing capacity deteriorates. Additionally, on the assumption that the road surface soundness level stays within

i $(i = 2, \dots, I)$, an average required time, $E[T](s|i)$, from road construction to the time when the load bearing capacity proceeds to s $(s = 2, \dots, S)$ can be defined as follows:

$$E[T](s|i) = \sum_{k=1}^{s-1} \frac{1}{\lambda^k(i)} \tag{3.63}$$

Formula (3.63) indicates the average required time from the service start time to the time when the load bearing capacity proceeds to s $(s = 2, \dots, S)$. We designate this as the load bearing capacity performance curve. Similarly, on the assumption that the load bearing capacity stays within s $(s = 2, \dots, S)$, $E[T](i|s)$ from the time of road surface repair to the time when the soundness level decreases to i $(i = 2, \dots, I)$ can be defined as follows:

$$E[T](i|s) = \sum_{l=1}^{i-1} \frac{1}{\mu^l(s)} \tag{3.64}$$

We designate this the road surface soundness level performance curve.

Table 3.10 Parameter estimation results (load bearing capacity)

Load bearing capacity: s	Road surface soundness level									
	(i = 1)		(i = 2)		(i = 3)		(i = 4)		(i = 5)	
	$E[\lambda^s(i)]$	$ET^s(i)$ (year)	$E[\lambda^s(i)]$	$ET^s(i)$ (year)	$E[\lambda^s(i)]$	$ET^s(i)$ (year)	$E[\lambda^s(i)]$	$ET^s(i)$ (year)	$E[\lambda^s(i)]$	$ET^s(i)$ (year)
1	0.114	8.754	0.194	5.160	0.318	3.140	0.366	2.736	0.408	2.454
2	0.115	8.669	0.196	5.110	0.322	3.110	0.369	2.709	0.412	2.430
3	0.162	6.168	0.275	3.635	0.452	2.212	0.519	1.928	0.578	1.729
4	0.328	3.053	0.556	1.799	0.913	1.095	1.048	0.954	1.169	0.856

Note: For each soundness level, $E[\lambda^s(i)]$ shows hazard rate of the load bearing capacity, s, for the case of road surface soundness level i, and $ET^s(i)$ shows expected life (year) of s with i as a given condition.

Table 3.11 Parameter estimation results (road surface soundness level)

Road surface soundness level: i	Load bearing capacity							
	(s = 1)		(s = 2)		(s = 3)		(s = 4)	
	$E[\mu^i(s)]$	$ET^i(s)$ (year)	$E[\mu^i(s)]$	$ET^i(s)$ (year)	$E[\mu^i(s)]$	$ET^i(s)$ (year)	$E[\mu^i(s)]$	$ET^i(s)$ (year)
1	0.093	10.749	0.224	4.457	0.304	3.287	0.373	2.682
2	0.292	3.427	0.704	1.421	0.954	1.048	1.170	0.855
3	1,999	0.500	4.821	0.207	6.537	0.153	8.013	0.125
4	3.810	0.263	9.187	0.109	12.457	0.080	15.269	0.065
5	6.319	0.158	15.239	0.066	20.663	0.048	25.329	0.039

Note: For each soundness level, $E[\mu^i(s)]$ shows hazard rate of road surface soundness level i in the case of load bearing capacity s, and $ET^i(s)$ shows expected life (year) of i with s as a given condition.

Figure 3.4 shows the road surface soundness level performance curve calculated with the estimated compound hidden Markov pavement deterioration model, and Figure 3.5 shows the

load bearing capacity performance curve. For example, we determined that the required time for the load bearing capacity to reach application limit $s = 5$ is approximately 26.6 years when the given condition is road surface soundness level $i = 1$.

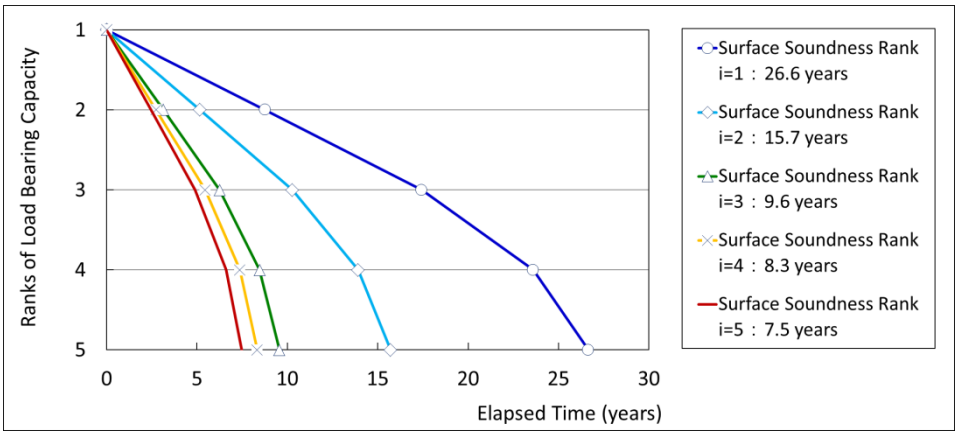


Figure 3.4 Performance curves of road surface soundness level

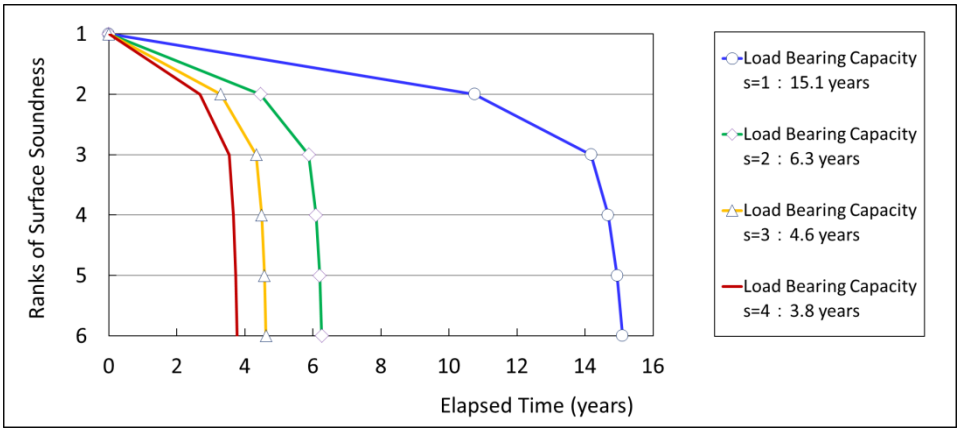


Figure 3.5 Performance curves of load bearing capacity

On the other hand, the deterioration speed of the load bearing capacity accelerates as the road surface soundness level deteriorates. To be more precise, when comparing each load bearing capacity performance curve in the case of road surface soundness levels $i = 2$ and $i = 5$, we determined that the former expected life is approximately 15.7 years, the latter expected life is approximately 7.5 years, and the deterioration of the latter case is approximately 2.1 times faster than that of the former. On the other hand, when comparing the performance curves of the road surface soundness level and load bearing capacity, we found that the expected life, up to the application limit, of the road surface soundness level is shorter than that of the load bearing capacity. Additionally, we found that the deterioration speed of the road surface soundness level increases as the load bearing capacity deteriorates (as the s value becomes larger). Concretely, when comparing each road surface performance curve in the case of load bearing capacities $s = 2$ and $s = 4$, we found that the expected lives of the former and latter

cases are approximately 6.3 and 3.8 years, respectively, and the deterioration speed of the latter case is approximately 1.7 times faster than that of the former case. Compared to the influence of the load bearing capacity on the deterioration speed of the road surface soundness level, the influence of the soundness level on the deterioration process of the load bearing capacity is smaller. Instead of the existing hierarchic hidden Markov deterioration model, when we use the compound hidden Markov pavement deterioration models, we can acquire information about the influence of the deterioration of the surface layer on the deterioration speed of the load bearing capacity. However, in actual pavement maintenance and management operations, it is unrealistic to assume that either the load bearing capacity or the road surface soundness level will always maintain a constant level of soundness while that of the other is deteriorated. Therefore, on the assumption that road surface soundness level i is immediately corrected and restored to 1 when it reaches a certain value, we determined the load bearing capacity and road surface soundness level performance curves by considering the influences of both deterioration processes. When the expected life, ET^s, of load bearing capacity $s-1$ is $\tau^s_{s,i,\Phi}$, we have the following formula:

$$\tau^s_{s,i,\Phi} = \frac{1}{\lambda^{s-1}(i^*)} +$$

$$\begin{cases} \displaystyle\sum_{l=i^*}^{i-1}\left\{-\frac{1}{\lambda^{s-1}(l)} + \frac{exp\{-\lambda^{s-1}(l+1)(\tau^s_{s-1,l+1,\Phi} - \tau^s_{s-1,l,\Phi})\}}{\lambda^{s-1}(l+1)}\right\} \\ \displaystyle\prod_{z=i^*}^{l} exp\{-\lambda^{s-1}(z)(\tau^s_{s-1,z-1,\Phi} - \tau^s_{s-1,z,\Phi})\} \\ \qquad (in\ the\ cases\ of\ \Phi = \Phi^*) \\ \displaystyle\sum_{l=i^*}^{l'-1}\left\{-\frac{1}{\lambda^{s-1}(l)} + \frac{exp\{-\lambda^{s-1}(l+1)(\tau^s_{s-1,l+1,\Phi^*} - \tau^s_{s-1,l,\Phi^*})\}}{\lambda^{s-1}(l+1)}\right\} \\ \displaystyle\prod_{z=i^*}^{l} exp\{-\lambda^{s-1}(z)(\tau^s_{s-1,z-1,\Phi^*} - \tau^s_{s-1,z,\Phi^*})\} \\ \displaystyle\sum_{l=1}^{i-1}\left\{-\frac{1}{\lambda^{s-1}(l)} + \frac{exp\{-\lambda^{s-1}(l+1)(\tau^s_{s-1,l+1,\Phi} - \tau^s_{s-1,l,\Phi})\}}{\lambda^{s-1}(l+1)}\right\} \\ \displaystyle\prod_{z=1}^{l} exp\{-\lambda^{s-1}(z)(\tau^s_{s-1,z+1,\Phi} - \tau^s_{s-1,z,\Phi})\} \\ \displaystyle\prod_{z=i^*}^{l'-1} exp\{-\lambda^{s-1}(z)(\tau^s_{s-1,z+1,\Phi^*} - \tau^s_{s-1,z,\Phi^*})\} \\ \qquad (in\ the\ cases\ of\ \Phi \neq \Phi^*) \end{cases}$$

where i^* is the road surface soundness level and Φ^* is the number of individual surface repairs for the case in which the load bearing capacity reaches $s-1$. The following formula is established as notational rules:

$$\begin{cases} \sum_{l=i^*}^{i-1} \left\{ \frac{1}{\lambda^{s-1}(l)} - \frac{exp\{-\lambda^{s-1}(l+1)(\tau_{s-1,l+1,\Phi}^{s} - \tau_{s-1,l,\Phi}^{s})\}}{\lambda^{s-1}(l+1)} \right\} = 0 \\ \qquad\qquad (in\ the\ cases\ of\ i = i^* and\ \Phi = \Phi^*) \\ \lambda^s(i) = \lambda^s(1) \quad (in\ the\ case\ of\ i = I') \end{cases}$$

Similarly, $ET^i(\Phi)$, which is an expected life of road surface soundness level i for the case in which the number of road surface repairs is Φ, is established as follows in the case of $ET^i = \tau_{s,i,\Phi}^{i}$:

$$\tau_{s,i,\Phi}^{s} = \frac{1}{\lambda^{s-1}(i^*)} +$$

$$+ \sum_{k=s^*}^{s-1} \left\{ - \frac{1}{\lambda^{i-1}(k)} + \frac{exp\{-\lambda^{i-1}(k+1)(\tau_{k+1,i-1,\Phi}^{i} - \tau_{k,i-1,\Phi}^{i})\}}{\lambda^{i-1}(k+1)} \right\}$$

$$\prod_{z=s^*}^{k} exp\{-\lambda^{i-1}(z)(\tau_{z-1,i-1,\Phi}^{i} - \tau_{z,i-1,\Phi}^{i})\}$$

s^* shows the load bearing capacity at the time when the load bearing capacity reaches $i-1$. The following formula is established as notational rules:

$$\begin{cases} \sum_{k=s^*}^{s-1} \left\{ - \frac{1}{\lambda^{i-1}(k)} - \frac{exp\{-\lambda^{i-1}(k+1)(\tau_{k+1,i-1,\Phi}^{i} - \tau_{k,i-1,\Phi}^{i})\}}{\lambda^{i-1}(k+1)} \right\} = 0 \\ \qquad\qquad (in\ the\ case\ of\ s = s^*) \\ \tau_{s,i,\Phi}^{i} = \tau_{s,1,\Phi+1}^{i} \quad (in\ the\ case\ of\ i = I') \end{cases}$$

Consequently, in consideration of the compound deterioration, Table 3.12 lists the expected life of load bearing capacity s for the case in which the road surface is repaired when road surface soundness level i reaches 5, and Table 3.13 lists the expected life in the case of road surface soundness level i and Ω road surface repairs. Additionally, based on Table 3.12 and Table 3.13, Figure 3.6 shows the compound performance curves of the load bearing capacity and road surface soundness level in consideration of the maintenance and management of the pavement with road surface repair. For comparison, this figure also shows the load bearing capacity and road surface soundness level performance curves for the case in which road surface soundness level i and load bearing capacity s are fixed at 1. Whereas the expected life of the load bearing capacity for the case in which i stays at 1 is approximately 26.6 years, the expected life of the load bearing capacity in the compound deterioration performance curve is approximately 22.9 years, 3.8 years shorter. On the other hand, whereas the expected life of the road surface soundness level for the case in which s stays at 1 is approximately 15.1 years, the expected life of the road surface soundness level in the compound deterioration performance curve is approximately 11.4 years even after service starts, 3.6 years shorter. With

the compound hidden Markov pavement deterioration model, we can express compound deterioration through consideration of the bidirectional influence on the pavement, and accordingly, more correctly analyze an optimal reconstruction time of asphalt layers. However, concerning the validity of the load bearing capacity performance curve from the aspect of pavement engineering, it is necessary to await sufficient future data accumulation.

Table 3.12 Expected life of load bearing capacity (compound)

Load bearing capacity: s	Expected life: ET^s (years)
1	8.754
2	15.822
3	20.663
4	22.873

Table 3.13 Expected life of road surface soundness level (compound)

Road surface soundness level: i	Expected life: ET^i (φ) (years)		
	Φ=0	Φ=1	Φ=2
1	9.712	4.435	3.287
2	1.421	1.048	0.894
3	0.207	0.153	0.125
4	0.109	0.080	0.065

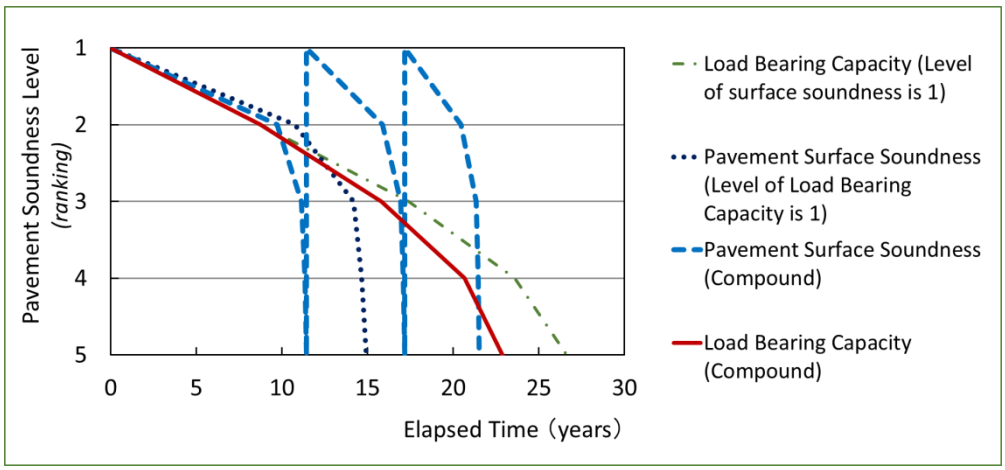


Figure 3.6 Compound performance curves

3.6.3 Test of Compound Deterioration Hypothesis

The compound hidden Markov model proposed in this study is based on the assumption that a compound deterioration hypothesis is established: the decrease in load bearing capacity accelerates the deterioration speed of road surface soundness level; simultaneously, the deterioration of road surface soundness level accelerates the decrease in load bearing capacity. In this study, we divide the compound deterioration hypothesis into two simple hypotheses: that the decrease in load bearing capacity accelerates the deterioration speed of road surface soundness level (Simple Hypothesis 1) and that the deterioration of road surface soundness level accelerates the decrease in load bearing capacity (simple Hypothesis 2). First, to test

Simple Hypothesis 1, we formulate a null hypothesis, $H_0^s (s = 2, \dots, S-1)$, and an alternative hypothesis, $H_1^s (s = 2, \dots S-1)$, as follows:

$$\begin{cases} H_0^s & : \quad \gamma_0^s > 1 \\ H_1^s & : \quad \gamma_0^s \leq 1 \end{cases} \tag{3.65}$$

When simple hypothesis model (3.65) cannot be rejected, we cannot deny the null hypothesis, which states that the deterioration speed of road surface soundness level in the case of load bearing capacity s $(s = 1, \dots, S-1)$ is faster than that of the road surface soundness level in the case of load bearing capacity $s = 1$. To test the null hypothesis based on MCMC, we define confidence interval $[\underline{\gamma_0}^{s,\varepsilon}, \infty)$ for level of significance ε with formulas (3.57a) and (3.57b), and standard sample order statistics $\underline{\gamma_0}^{s,\varepsilon}$ $(s = 2, \dots, S-1)$ as

$$\underline{\gamma_0}^{s,\varepsilon} = \arg\min_{\gamma_0^{s**}} \left\{ \frac{\#\left(\gamma_0^{s(r)} \geq \gamma_0^{s**} \quad \in \mathcal{M}\right)}{\bar{r} - \underline{r}} \leq \varepsilon \right\} \tag{3.66}$$

When value 1 is included in confidence interval $[\underline{\gamma_0}^{s,\varepsilon}, \infty)$ of γ_0^s, null hypothesis H_0^s can be rejected. Otherwise, H_0^s cannot be rejected. The current confidence interval for this time estimation result for the case in which level of significance ε is set to 0.05 is listed in Table 3.8. Value 1 is not included in the confidence interval for either soundness level. Therefore, we consider that H_0^s is not rejected, and the alternative hypothesis can be rejected. In the same way, to test Simple Hypothesis 2, we formulate a null hypothesis, $\widehat{H}_0^i$, and an alternative hypothesis, $\widehat{H}_1^i$, as follows:

$$\begin{cases} \widehat{H}_0^i & : \quad \beta_0^i > 1 \\ \widehat{H}_1^i & : \quad \beta_0^i \leq 1 \end{cases} \tag{3.67}$$

When simple hypothesis model (3.67) cannot be rejected, we cannot deny the null hypothesis, which states that the deterioration speed of the load bearing capacity in the case of road surface soundness level i $(i = 2, \dots, I-1)$ is faster than that of the load bearing capacity in the case of $i = 1$. To test the null hypothesis based on MCMC, we defined confidence interval $[\underline{\beta_0}^{i,\varepsilon}, \infty)$ for ε with standard sample order statistics $\underline{\beta_0}^{i,\varepsilon}$ $(i = 2, \dots, I-1)$ as:

$$\underline{\beta_0}^{i,\varepsilon} = \arg\min_{\beta_0^{i**}} \left\{ \frac{\#\left(\beta_0^{i(r)} \geq \beta_0^{i**} \quad \in \mathcal{M}\right)}{\bar{r} - \underline{r}} \leq \varepsilon \right\} \tag{3.68}$$

Similarly to the previous case, when value 1 is included in $[\underline{\beta}_0^{i,\varepsilon}, \infty)$ of β_0^s, $\overline{H}_0^i$ can be rejected; otherwise, $\overline{H}_0^i$ cannot be rejected. According to Table 3.9, concerning the current estimation results, we found that value 1 is not included for either soundness level. Therefore, we consider that $\overline{H}_0^i$ cannot be rejected, and the alternative hypothesis can be rejected.

3.6.4 Suggestions for Practical Application

The deterioration process of road pavement contains many kinds of uncertainty, and it is difficult to definitively forecast the deterioration process. We can verify the road surface deterioration condition with periodic road condition surveys. With the Markov deterioration model, it is possible to practically forecast the deterioration process based on the observed values in the actual road condition surveys [25],[26]. On the other hand, through long-term operation of the road, not only the road surface soundness level but also the overall pavement progressively deteriorate; and as a result, the pavement load bearing capacity decreases [27]. When the compound deterioration hypothesis is established, as the road surface soundness level is deteriorated, the decrease in the pavement load bearing capacity is accelerated; on the other hand, as the load bearing capacity decreases, the deterioration of the road surface soundness level is accelerated. Therefore, for a road section with deteriorated pavement load bearing capacity, by considering the optimal recovery time of the load bearing capacity to prevent the necessity of updating not only the road surface but also the pavement, we may be able to reduce the expected life cycle expenses [28],[29]. Additionally, in actual road management operations, the order of collecting observed data values of the road surface soundness level and pavement load bearing capacity is not always consistent. When a certain section shows serious deterioration of the road surface soundness level, surveys on this section may be operated as a priority for fear of reaching the application limit of the load bearing capacity. We can eliminate problems such as insufficient or absent observed values of either the road surface soundness level or the pavement load bearing capacity by using the compound hidden Markov pavement deterioration model. With this model, we can simultaneously estimate the load bearing capacity performance curve according to road surface soundness level, and the soundness level performance curve according to load bearing capacity. In the actual operations, by relative evaluation of the road surface soundness level and load bearing capacity performance curves, we can design an optimal time for rehabilitation of pavement structure and repair of its surfaces. Additionally, it is expected that the use of this study will contribute to the development of a screening method in which a road section to be investigated with an FWD survey is determined based on the road surface soundness level, and to medium to long-term planning aimed at updating the pavement.

3.7 CONCLUSION

In this study, to express the compound deterioration process in which two elements, the road surface and pavement load bearing capacity deterioration processes, affect each other, we developed the compound Markov deterioration model. Additionally, to eliminate the time inconsistency problem, which arose from the unavailability of information about surveys on simultaneous road surface and load bearing capacity deterioration, we proposed the compound hidden Markov deterioration model for considering the systematic missing data, and also presented the estimation method. With the application samples of an expressway's asphalt layers as an empirical study, initial results can be summarized as follows:

- When the road surface soundness level was high, the expected life of the load bearing capacity was 26.6 years, which decreased to 7.5 years in case the road surface soundness is at the most critical level.
- Due to the influence of pavement load bearing capacity, its surface soundness varies in a wide range from 15.1 to 3.8 years.
- Influence of the load bearing capacity on the road surface soundness level is larger than that of the road surface soundness level on the load bearing capacity.

Although these only represent results of research on specific road sections, we hypothesize that we can acquire universal knowledge by increasing the target sections to be analyzed, and particularly by verifying the consistency of the deterioration process of pavement load bearing capacity from the perspective of pavement engineering. Furthermore, we will discuss the following research tasks that should continuously be studied. The first is the practical utilization of these study results. For example, by using the relation between the expected life of the road surface and the scale parameters, we can develop a screening method that allows the extraction of road sections need FWD surveys. The second point is the application of the compound hidden Markov deterioration model to other deterioration phenomena. For example, the model can explain a deterioration phenomenon in which compound neutralization and internal salt damage accelerate the deterioration of concrete structures. Additionally, it can be expanded to load bearing capacity and cracking, or to multi-stage compound deterioration phenomena considering the influences of potholes. The third study is the development of a life-cycle evaluation method to simultaneously consider the deterioration of road surface soundness level and load bearing capacity, and further, to determine the optimal time for updating pavement structures and repairing road surfaces as well. Under the situation of increasing of aged road pavements, it is more demanded for strategic maintenance taking into consideration of both surface performance and strength of pavement structure.

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CHAPTER 4

Kyoto Model - A Platform for Pavement Management System

4.1 GENERAL INTRODUCTION

Road network of a country forms the backbone of national economy and requires huge financial investment that secures for the safe and efficient movement of people and goods at certain level of service. Economically feasible and technically sound decisions on planning and design, construction, maintenance, rehabilitation, and reconstruction of road pavements are essential for preserving road network in an acceptable performance (Uddin, W.)[1]. Therefore, pavement management system has been recognized as the most effective way for selection of cost-effective strategies and policies for pavement construction and maintenance under constraints of available resources.

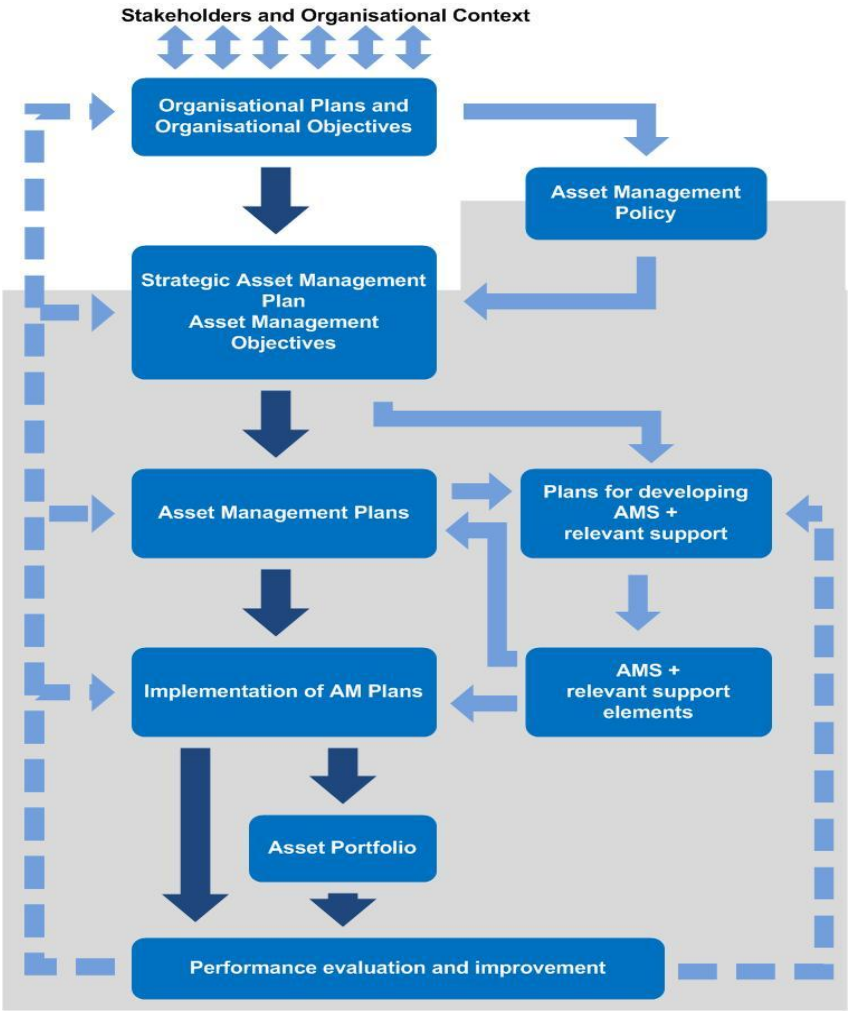
PMS concepts were introduced in the 1960s as a systematic manner to manage pavements that are designed to manage, plan, allocate budget, and schedule all the maintenance activities on pavements for road administrators (Sue McNeil, Pannapa Herabat)[2]. So far, there has been variety of researches, development as well as implementation of PMS in the world.

In more wider scope of asset management, an international standard for asset management known as ISO55000[3],[4] was established in January 2014 that is aimed at enabling the asset management PDCA cycle to function and achieve continual improvement of management (Figure 4.1).

This chapter presents a new PMS named as “Kyoto model” for pavement management which is consistent with the new international standard ISO55000 for asset management. The Kyoto model proposed in this study adopts methodology that estimates deterioration hazard model by using repair history data, and available datasets of pavement conditions periodically collected through its life cycle. The estimated performance curves explain the tendency that pavements had been deteriorated. As one of the support functions in PMS Kyoto model, improvement and evaluation based on past performance (Check and Action) in PDCA management cycle that plays an important role in asset management are available.

Kyoto model is a “Data Oriented Road Management System” to support PDCA cycle of road maintenance work. Kyoto model manages information acquired during the on-site

maintenance work, and provide with solutions by analyzing the field data, based on deterioration performance evaluation and Benchmarking analysis.



Source: ISO55000:2014 Asset Management - Overview, principles and terminology

Figure 4.1. Relationship between key elements of an asset management system (AMS)

4.2 BASIC APPROACH OF KYOTO MODEL

Since 1970, a variety of studies on the pavement management systems have been conducted. These studies are designed to reduce the life-cycle cost of road pavement at both project level and network level (Hass et al, 1994[5]; Hudson et al, 1997[6]; Hera bat et al, 2002[7]). Take example in United States, developed PMS has been configured as a system of centering and uniting in data banks for all activities related to pavement organically (Kasahara, 2005[8]). As in Japan, the studies on PMS have been positively executed; for instance, Jido et al (2004)[9] developed a pavement management accounting system (PMAS) for the road administrators of

local governments to execute rational repair by analyzing road pavement asset management information system.

HDM developed by the World Bank is recognized as the de-facto standard of pavement management system that widely open to public user in the world (PIARC, 2006[10]). The latest version, HDM-4, has been used as a support system for road development and maintenance plan mainly in the developing countries. HDM-4 plays the role of an analytical tool of PMS which supports economic analysis such as study on budget allocation, financial condition evaluation and road investment evaluation.

However, using HDM-4 for analysis, calibration of deterioration prediction model by using an enormous amount of data input concerning a variety of data related such as regional characteristic, weather, and traffic data should be conducted (Aoki, 2011[11]). In fact, it seems to be practically impossible to collect input information of those related data to operate PMS that aims at rationalization of pavement maintenance work. Therefore, the HDM-4 cannot be operated in optimal way due to the poor data preparation that seems to be very common in many developing countries. The deterioration prediction model installed in HDM-4 predicts deterioration speed by adding road characteristic data (calibration data) as standard for deterioration prediction model and without using the archive data from actual investigation of road pavement maintenance targets. However, deterioration process of road pavement is very complex, and various factors relate mutually, a certain specific pavement section is deteriorated. A difficult factor to observe such as material characteristics, conditions during construction besides observational factors such as traffic and meteorological conditions might have a great impact for deterioration performance of pavement. Moreover, different conditions of pavement section, different performance after putting into operation. Therefore, it is difficult to specify deterioration factor and also there is diversity in the factor in deterioration in the road pavement. Even if enormous input information for deterioration prediction is collected so that HDM-4 may demand, it is impossible to express deterioration performance in pavement section concerned.

The Kyoto model proposed in this study adopts methodology that estimates deterioration prediction models (deterioration hazard models) by using repair history data, and time-series data from pavement condition surveys. As the result of model operation using available data, estimated performance curves will show the deterioration process including past deterioration for target pavement sections along its life cycle. In PMS, improvement and evaluation based on past performance (Check and Action) play an important role. An actual performance of road pavement is expressed in the archived data such as repair history data and results of periodical pavement surveys. In using deterioration prediction model that modeled rule of this archive data, the deterioration process in the future is assumed equal to past performance of concerned road pavement. As for the Kyoto model, a basic concept of the deterioration prediction model is different with de-facto model of HDM-4. In addition, in Kyoto model, by

doing evaluation using past deterioration performance, more meaning feedbacks and proper recommendations for the next plan can be made to formulate practical action plans.

4.3 THE STRUCTURE OF KYOTO MODEL

Pavement management system Kyoto model proposes regularly monitors by comprehensive perspective of maintenance works at management cycle of maintenance state and budget execution intended for such works, and has hierarchical structure of strategic management cycle to point out improvement of maintenance works by policy evaluation. The pavement maintenance can be expressed by hierarchical management cycle, and the entire of work process is modeled by using logic model. Meanwhile, various data groups necessary for pavement maintenance are used as an evaluation index of logic model for policy evaluation as well as being reference for decision making of maintenance works.

The whole process of pavement management can be described by hierarchical management cycle PDCA. Figure 4.2 indicates the structure of pavement management.

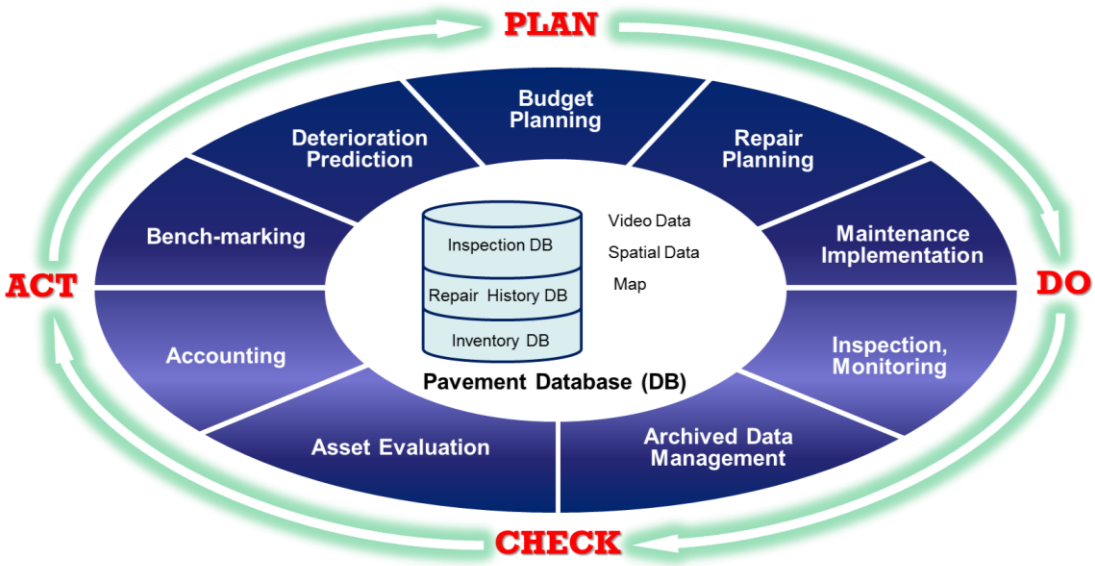
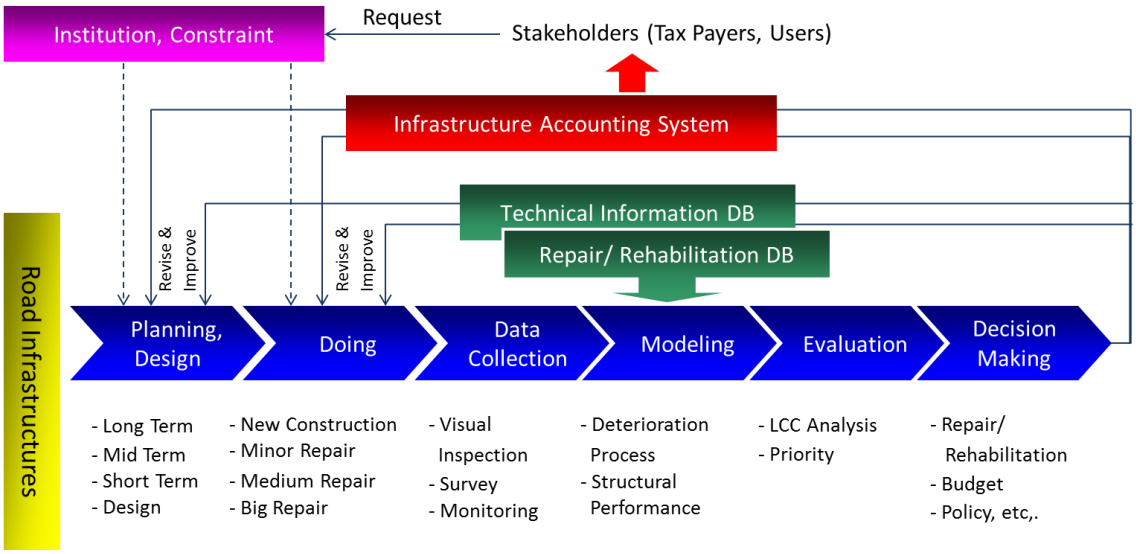


Figure 4.2 Relevant Activities within PDCA Cycle for Asset Infrastructure Management

In road infrastructure management, PDCA can be explained in more practical manner of its components, activities and relationships as shown in Figure 4.3.



(Source: Kaito, K., 2013)

Figure 4.3 General PDCA Cycle in Road Infrastructure Management

The process of pavement management can be divided into three levels among the different decision making stages. At the strategy level (i.e. long-term planning), predictions in the long term about optimization of the policy of the inspection, repair, and replacement and budget transition, and required budget are supplied. At the tactical level (i.e. single fiscal year network planning), the distribution of the budget for each pavement group and the candidate for management is considered according to fiscal year budget as determined on the strategy level. At the project level, a detailed design for a repair section is performed.

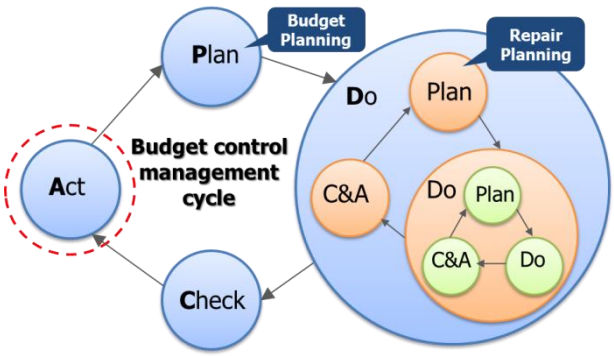


Figure 4.4 Hierarchical PDCA Cycle in Concept

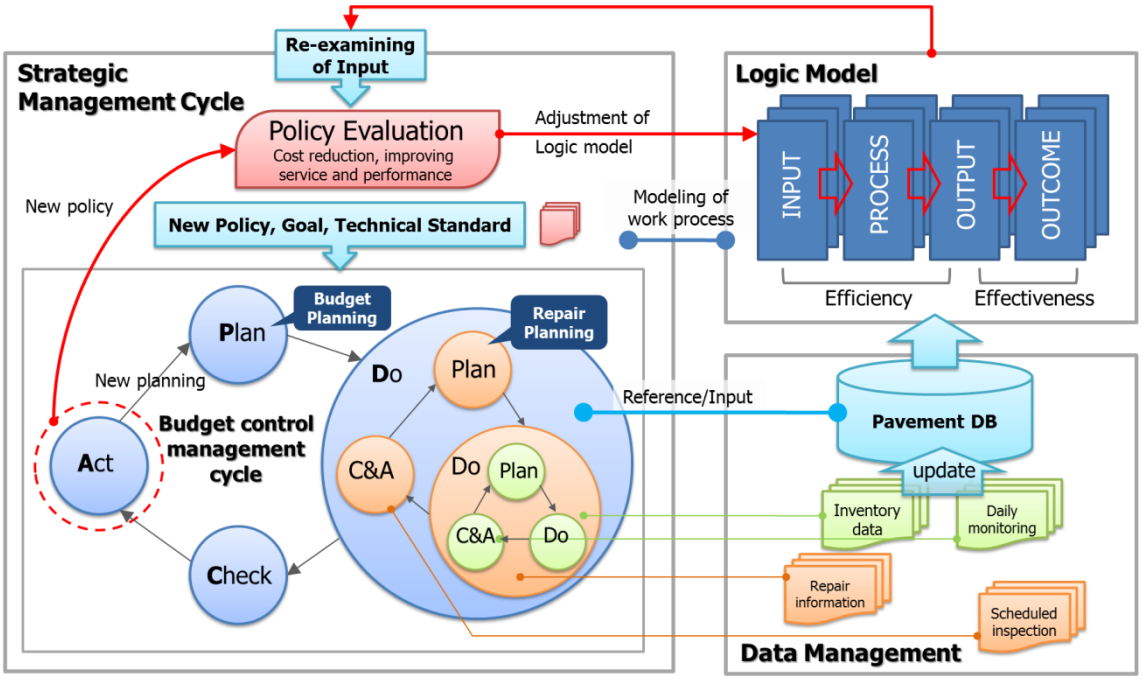


Figure 4.5 The whole structure of Kyoto model

4.4 ESSENTIAL COMPONENTS OF KYOTO MODEL

4.4.1 Logic Model

In the reality of road works, there are various activities relevant to pavement maintenance, for example, inspection (daily patrol, periodic inspection, urgent check), maintenance works and budget planning. To tackle the problem of these activities, there is always a hypothetical logic that explains what outcomes are expected by execution pavement activities. As consequently, those individual activities can logically describe a causal relationship leading up to maintenance targets and outcomes originated from implementation of those activities (Sakai et al, 2007) [13] .

A logic model is systematic and visual way to integrate the relationship among the resources or inputs, process of implementation necessary activities, outputs or outcomes, and impact. The logic model is a beneficial evaluation tool that facilities effective program planning, implementation and evaluation (W.K Kellog, 1998) [14]. The logic model takes the role to observe the management cycle of PDCA in road pavement maintenance.

The result that executes the work based on the logic model is quantitatively evaluated as an output index, and as a result, target of input improvement is clarified by analyzing contribution level of individual input to the entire works, as well as identifying the input with remarkable or poor contribution. These information shall be integrated into next planning.

Policy evaluation based on the logic model requires regularly monitoring, and maintenance information is required in evaluation and calculation output index and outcome indicators. Information that should be acquired in maintenance work is decided by evaluation item of the logic model.

The logic model is modeled to reach "Result (outcome)" finally where specific activities pass a certain process (Sakai et al, 2007) [13]. As shown in Figure 4.5, execution of logic model help to achieve output and outcome from inputs based on the processing procedure. Brief description of elements in logic model is shown in Table 4.1.

Table 4.1 Elements of Logic model

Elements	Description
INPUT (Resource and Activities)	Resources and activities (budget and manpower, etc,.) being mobilized to perform the operation
PROCESS (Activities and Procedures)	Set of necessary activities and procedures to realize output based on mobilized input
OUTPUT (Results)	Results obtained from personnel activities
OUTCOME (Management Objectives)	**Intermediate outcomes** (results): Short-term outcomes that may result from activities and outputs.
	Final outcomes (Management Objectives): Final management objectives. It generally takes a long term to achieve and may be affected by an external factor.

Figure 4.6 shows one example of formulation a logic analysis for the target or final outcomes of (i) traffic safety and riding comfort securing to provide convenience to road users including reduction of vehicle operation cost, and (ii) prolonging pavement service life as the requirement of road administrator to minimize LCC. In order to approach to the objectives, current situation must be clarified to identify the necessary for taking right actions.

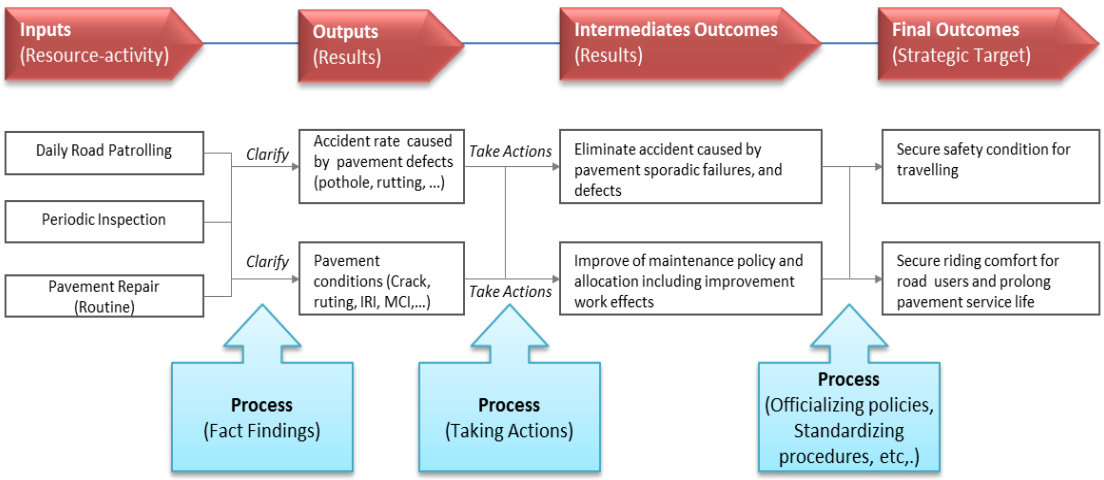


Figure 4.6 Example of Logic Model

As for reconsidered, pavement routine maintenance is implemented in order to ensure safety of road users. Causal relationship in pavement routine maintenance is necessary for analysis. As a result, it is important to arrange the objectives and measures of entire pavement maintenance work systematically. In this pavement logic model, the inputs mainly include pavement routine maintenance and periodic inspection, and the outcomes include safety and riding comfort of road users, and longer service life for pavement. Causal relationships in the intermediate processes are systematically described by using intermediate outcome and output indicators, as of enable of quantitative evaluation as much as possible. A policy evaluation model is established to evaluate the relationship between inputs and outcomes.

A pavement logic model is a tool for activities of maintenance works such as maintenance implementation, budget, and inspection (inputs) to measure influence of achieving final target of pavement management as indicated in safety and securing comfort ability. Furthermore, it is a tool to monitor achievement situation of the result (outcomes) regularly, and reviews the inputs in relating to improving the entire system.

The object that is evaluated and improved according to the pavement logic model shows a technical indicator such as patrol frequency, inspection parts and data items, and introduction new methods of maintenance for instance. These objects come in analyzes various data generated by continuing implementing the process of routine pavement management work, arranges the review of the inputs as an indicator, and disseminates to maintenance field.

4.4.2 Pavement Database

It is quite natural to recognize that database is always the heart of any management system. As for road asset management and road & traffic operation management system, its necessity is more indispensable than the other management systems. Because, in general, road network expands in a large extent regardless of variation of natural conditions in geography, topography, hydrology, environment, etc., and deterioration progression of road facilities including pavement depends on such natural and environmental conditions greatly. Moreover, in management of road assets and road & traffic operation, a large number of agencies and organizations are involved that makes using a common and consistent data becomes necessary for efficient and smooth implementation. Therefore, road database is a very significant component for road administrators for various purposes including road asset management, deterioration prediction, maintenance planning, decision making for policy formulation, and operation & management of road & traffic.

Many kinds of data should be requested to integrate into the database depending on the functions and demands of the system. In Kyoto model, data can be grouped into three main data components of inventory data, maintenance history data, and inspection data. Pavement

inventory data is formulated to make the platform that is rather static and stable to record and register other more dynamic data of road pavement conditions and maintenance activities in the two later components. Changes of road pavement condition including traffic should be registered to inventory component for updating based on routine patrolling and monitoring.

Inventory data in pavement database is rather simple that normally comprises of some main data items identifying the location of pavement sections (km-post chainage, direction, lane code, and dimensions of sections such as length and width), administrative information to show the road management agencies.

In order to forecast the demand for repair in the future, a method has been devised using time series data from the results of road pavement repairs in the past and surveyed pavement conditions. Being a data-oriented system, Kyoto model is integrated with sophisticated deterioration forecasting model to minimize requirement on data preparation. For pavement deterioration prediction, only two time-series datasets of pavement conditions are requested that consist of three pavement performance indicators: cracking ratio, rutting depth, and roughness in IRI. In the case that only one time-series data of pavement condition is available, the minimum deterioration model can be built by supplementing maintenance history information of the recent repair time or year of new construction completion. In more common operation, other data of traffic volume and pavement structure and thickness should be used to conduct further evaluation such as factor analysis to identify the main causes of pavement deterioration or distresses as well as benchmarking evaluation.

Figure 4.7 Pavement sections with similar conditions

In road maintenance, time series datasets are also necessary to identify the repair priority for road pavement. In many cases, the conditions of pavement sections that belong to the group of candidate repair sections are quite similar that make the selection of right sections for repair becomes difficult especially under the condition of maintenance budget constraint. However, if the time series datasets are available, by simple comparison the deterioration trend, more critical sections with the higher priority for repair can be identified. As shown in Figure 4.7, Figure 4.8, even both section A and section B have the similar surface crack ratio at the current

inspection but the deterioration process of section A is more faster than section B that requires the higher priority for section A needs being repaired.

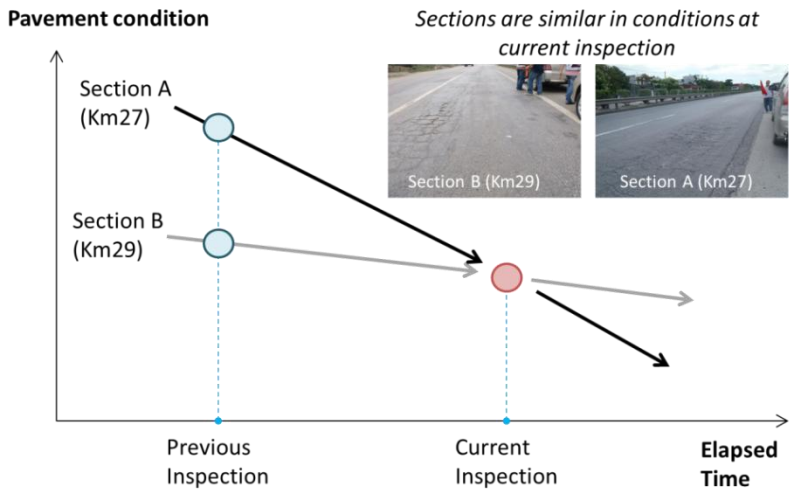


Figure 4.8 Utilization of time-series datasets to identify the repair priority

Over the time of road operation, database should be enriched with huge data especially time-series data. In Kyoto model, there is no limitation of data utilization. The more time-series data, the better outcomes including pavement deterioration curves; the richer the data, the more diversified results. If more characteristic data such as natural conditions, pavement technologies are available, the analysis results become more informative and useful to support for taking right action on road management and maintenance.

Based on users' requirements on data, data can be arranged in simple manner or complicated one. In general, it is expected to approach to a real database that works like a data machine to provide the best utility for users with the full functions of importing, processing, displaying and exporting also. In the case of application WEB based operation, data should be managed in a WEB database to support Web applications or services including the enhancement of displaying function on WEB GIS map for visualization. Moreover, WEB database with multi-users operation mechanism is regarded as one solution for accelerating formulation of a road database in developing countries. In this case, beside the technological issues, it is also strongly recommended to enhance institutional issues such as the mechanism and policy of data formulation, updating including approval of a new cost item for data collection.

4.4.3 Plan - Do - Check - Act in Strategic Management Cycle

4.4.3.1 Functions

Strategic management cycle supports to review maintenance method regularly, to set technological standards, target and new policies to achieve improvement of performance (making long-lived) and improvement of service as well as cost reduction to apply in

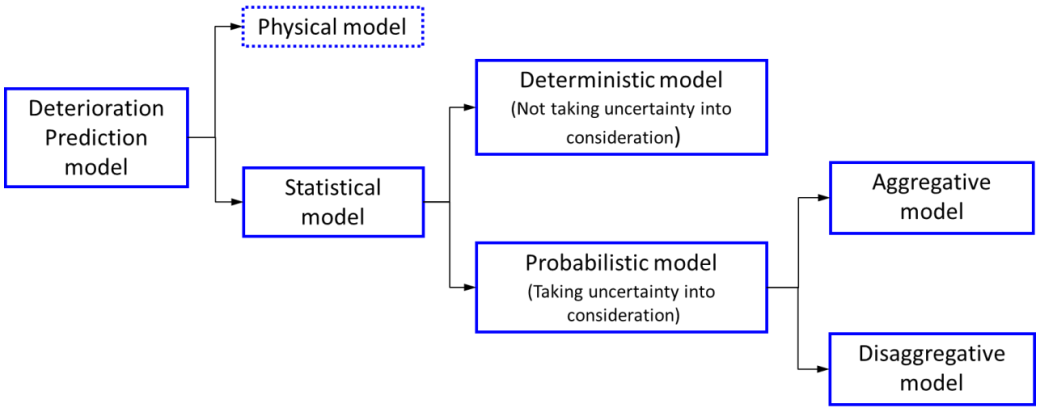
maintenance works. Planning, Doing, Checking and Taking Action are the four main components of strategic management cycle.

Planning of pavement maintenance works can be divided into three levels of strategy or long-term planning, tactical or program planning for mid-term, and short-term planning at the project level. The third level of planning focuses on formulation detail annual plans with the detail information of repair sections and corresponding maintenance works for implementation. Strategy planning provides long vision of the condition of road pavement infrastructure in the future that should be used for the road sector to set up strategy of road development and preservation. Maintenance and management scenarios are usually proposed for analysis in the long-term planning. In order to set up action plans for the goal of the strategy, budget preparation for each phase in mid-terms should be identified for mobilization.

Before approaching to planned and preventive maintenance, after-the-fact maintenance has been widely applied especially in developing countries under the critical condition of lacking data. For after-the-fact maintenance, the maintenance planning is just corrective that focuses on proposing the list of priority sections with distresses had been occurred for repairing. In this case, it seems to be impossible to approach to optimal strategy of road maintenance and management because the concept of life cycle cost cannot be introduced in the case of data lacking. On the contrary, in planned and preventive maintenance, maintenance works should be planned for not only deteriorated sections but also potential sections of deterioration based on the forecasted image of pavement conditions in the future.

4.4.3.2 Deterioration Forecasting Model

There are varieties of the methods for predicting deterioration of physical infrastructure assets over time. Kaito, et al. (2007) [15] broadly classified these methods into two groups: (i) physical methods based on the mechanical deterioration mechanisms of structural members, and (ii) statistical methods based on inspection data carried out in the past. Selection of deterioration prediction methods should be decided based on the objectives of deterioration prediction.



(Source: Kaito, K., 2007)

Figure 4.9 Classification of Statistical Methods for Deterioration Prediction

In the field of pavement engineering, there are some typical accelerated testings that belongs to physical model to understand behaviour mechanisms of pavement structure till its failure under simulated testing models in the laboratory. Figure 4.10 shows the testing model to identify rutting resistance of asphalt concrete specimens.

a) Hamburg Wheel Tracking Test

b) Tested Sample

Source: http://www.epc.com.hk/product_en_2889.html

Figure 4.10 Hamburg Wheel Tracking Test for Rutting Resistance of Asphalt Concrete

In practice, physical methods are often selected for decision making about micro-level issues such as the detailed lifetime estimation of individual infrastructure and its repair or reinforcement tactics, while statistical methods are often adopted for decision making about macro-level issues such as the budgetary management of the whole infrastructure systems and their maintenance strategy in the future. Of these, Kyoto model is data oriented system so the latter approach had been selected to develop model for pavement deterioration prediction.

An abundance of statistical methods for deterioration prediction based on visual inspection data have been proposed as shown in Figure 4.9. Such classification is made according to whether or not these methods take into account the uncertainty of the deterioration processes, and whether estimation procedures are aggregative or disaggregative. Here, aggregative methods mean ones predicting deterioration at macro levels by collecting all the visual inspection data of infrastructures and implementing some kind of averaging. On the other hand, non-aggregative methods mean ones predicting deterioration at micro levels, looking at each characteristic by using the visual inspection data of individual infrastructure.

As examples of deterministic methods which do not take uncertainty into consideration, there are the studies of Yanev, B. (1997)[16] and Kaito, et al.(2003)[17]. Yanev, B. (1997)

calculated a deterioration curve for bridges based on the visual inspection data of bridges in New York City. However, he also pointed out that the method would predict gentler deterioration than the actual deterioration if the complete inspection history of bridges is not considered. With regard to this application limit, other literature (Abed-Al-Rahim, I.J. and Johnston, D.W., 1995) [18] also reports the same conclusion. On the other hand, Kaito, et al. (2003) focused on the deterioration rates obtained from the data of visual inspections carried out two times for the same samples of infrastructures, and proposed a method for calculation its average deterioration curve. The advantage of this method is that even if the past inspection history is not available but there are two time-series inspection datasets, it is possible to solve the above-mentioned application limit to predict deterioration.

Methods applying the Markov Chain Model are typical of the probabilistic methods that take uncertainty into consideration. As aggregative prediction methods using Markov transition probabilities, there are the studies by Akaishizawa, N., et al. (2001) [19] and so on. However, these studies did not take into account the non-uniformity of inspection intervals that the authors point out, and carried out aggregate calculation with the implicit assumption that the inspection intervals are constant. On the other hand, as a disaggregative method, there is the study by Tsuda, Y. et al. (2005) [20], who described deterioration processes with a multi-stage exponential hazard model, and proposed a method for estimating Markov transition probabilities. The merit of this method is that it is possible to calculate a deterioration curve by considering the unique characteristics of structural members and environmental conditions. In addition, as far as the authors know, this study overcomes the effect of non-uniformity of inspection intervals. However, Tsuda, et al. point out that many data are required to estimate Markov transition probabilities with retaining certain accuracy. Furthermore, in order to predict deterioration of infrastructures at macro levels, which is the target of Kyoto model, it is ultimately necessary to perform aggregate calculations. Taking the above-mentioned issues into consideration, this study focuses on the aggregative methods, which are inferior to the exponential hazard model-based disaggregative methods in the precision of their estimations, but highly feasible and easy to understand conceptually, and then attempts a solution to the non-uniformity of inspection intervals. Then, in Kyoto model, a Markov chain model that takes into consideration the non-uniformity of inspection intervals has been proposed to develop a methodology for estimating transition probabilities using Markov hazard model as shown in following equations. More details about deterioration forecasting including Benchmarking analysis in Kyoto model is described in Chapter 2, Chapter 5 and Chapter 6.

$$\pi_{ii} = exp(-\theta_i Z) \tag{4.1}$$

$$\pi_{ii+1} = \frac{\theta_i}{\theta_i - \theta_{i+1}}\{-exp(-\theta_i Z) + exp(-\theta_{i+1} Z)\} \tag{4.2}$$

$$\pi_{ij} = \sum_{k=i}^{j} \prod_{m=i}^{k-1} \frac{\theta_m}{\theta_m - \theta_k} \prod_{m=k}^{j-1} \frac{\theta_m}{\theta_{m+1} - \theta_k} \, exp(-\theta_k Z) \tag{4.3}$$

$$\pi_{iJ} = 1 - \sum_{j=i}^{J-1} \pi_{ij} \tag{4.4}$$

The condition state of a pavement i is defined by a rank J representing a state variable $(i = 1, \dots, J)$. θ_i is the hazard function or hazard rate describing speed of deterioration transition from state i to state $(i + 1)$ or the slope of deterioration performance curve (Figure 4.11).

π_{ij} is the component probability the Markov transition probability matrix in equation (4.5) in that the condition state of road facilities such as pavement transits from state i to state j after one transition step ($j \geq i$).

$$\boldsymbol{\Pi} = \begin{pmatrix} \pi_{11} & \cdots & \pi_{1J} \\ \vdots & \ddots & \vdots \\ 0 & \cdots & \pi_{JJ} \end{pmatrix} \tag{4.5}$$

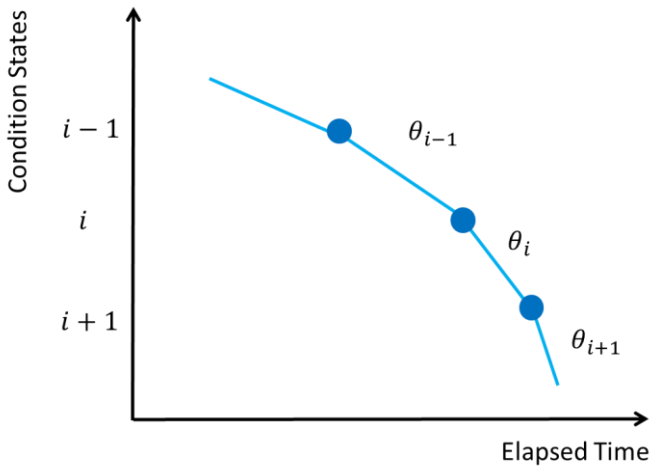


Figure 4.11 Hazard rate and deterioration performance curve

Transition of condition state of road facilities including pavement could be visualized using Markov transition probability matrix as shown in Figure 4.12. The transition shows how road facilities deteriorate through its service life since the brand-new state to the critical level of service that make the estimation of pavement life expectancy becomes possible. Moreover, the difference among transition of each state is also so meaning for road administrator to

understand the significance of road maintenance and setting priority for repair. As shown in Equation 4.1, 4.2, 4.3, and 4.4, transition probabilities only depend on parameters that are estimated using time-series data. There exists no calibration at all by users or analysts. Therefore, Kyoto model is understood as data-oriented pavement management system.

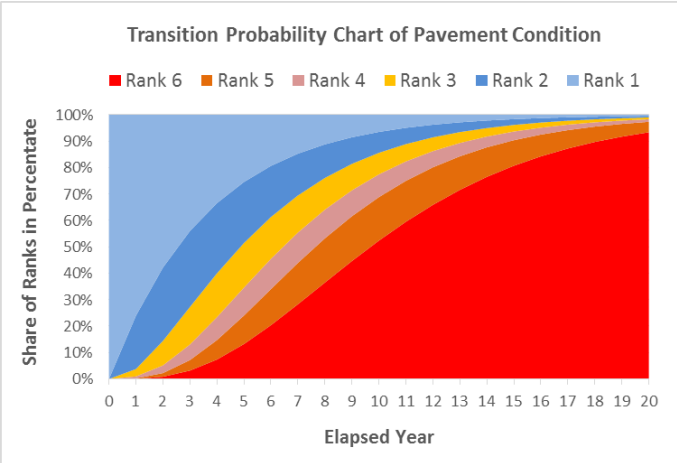


Figure 4.12 Transition of condition state of road pavement

The deterioration forecasting methodology in Kyoto model is quite sophisticated to makes it work as data-oriented model and extremely support for road management including any initial effort of data preparation. Figure 4.13 shows the performance curves of pavement in National Highway No.2 in Vietnam predicted by using Kyoto model. The road was upgraded fully and put into operation in 2008 with the total length of 22 kilometers and cross-section of 4 motorized lanes for both directions. With the first effort of the management company in pavement condition survey in 2014 for just 88 kilometers.lane, informative and useful results are made to support them approach to planned and strategic maintenance.

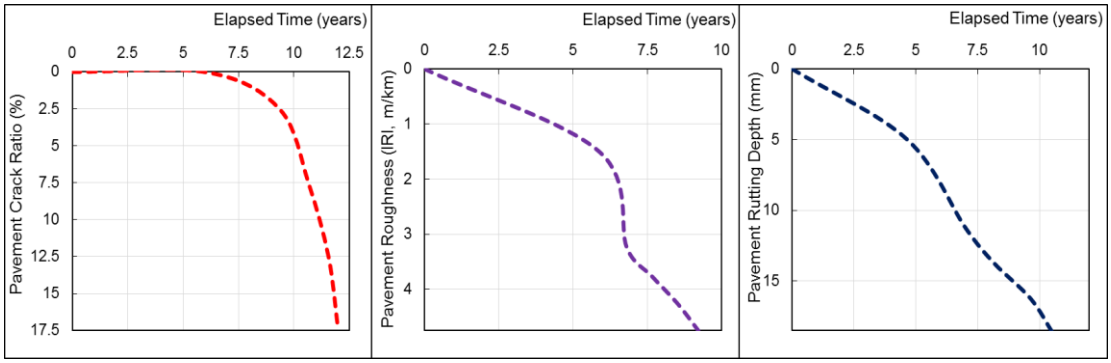


Figure 4.13 Deterioration progression of pavement in NH2 under condition of limited data

4.4.3.3 Benchmarking Analysis and Post-Evaluation

In designing the pavement surface structure, designers need to propose various surface alternatives taking consideration of different issues of local materials, available technologies, operation conditions, etc,. for analysis to select the proper structure, which is usually optimal in

term of expected cost analysis. In many practical situations, several pavement surface technologies or structures can be proposed for construction. As the matter of course, in reality it can be found that each pavement structure shows its own deterioration process that is different from others. Moreover, under the different operation conditions in terms of traffic loading, climate conditions, geotechnical and subgrade conditions and so forth, the same pavement also shows its difference in performance or deterioration that leads to the variety of its service life. Based on the available data, Benchmarking analysis is integrated in Kyoto model to support clarifying these above difference of pavement performance or deterioration for its various groups in pavement technologies, operation conditions, maintenance performance, etc.

Local mixture Markov deterioration hazard model is integrated into Kyoto model employing heterogeneity factor ε to express the inhomogeneous sampling population. Regarding road pavement, it is assumed that the entire network of roads in analysis consists of K groups in total of road sections. The individual group k $(k = 1, \dots, K)$ consists of S_k road sections with the heterogeneity factor ε^k, which infers the change of hazard rate of condition state i $(i = 1, \dots, J-1)$ with respect to the pavement section $s_k (s_k = 1, \dots, S_k)$. If the value of heterogeneity factor ε^k for group k is higher than that of other groups, the group k performs a faster deterioration than other groups.

Benchmarking process supports the designers, road managers and administrators to seek for the best managerial practices. In another word, it is the comparison in term of deterioration process among available technologies on infrastructure. That's also the needed information for post-evaluation to identify the best solution among many trials of pavement technologies. Useful recommendations can be made to realize or standardize selected technologies.

Based on available data of road pavement conditions collected in 2001 and 2004 in Vietnam, Nam et al., (2009) [21] had studied the difference of pavement deterioration in different regions characterizing by regional characteristic variables such as climate conditions, temperature, geotechnical and hydrological conditions and so forth. As shown in Figure 4.14, along the nation, six regions are defined that formulate six groups of pavement sections belong to each region. The ranking system or pavement condition state was introduced to evaluate pavement conditions. Pavement in state 1 has the best condition. Conversely, pavement in the worst condition is assigned condition state 5.

Exponential hazard model is applied to estimate the Markov transition probability. Further, the heterogeneity factor of individual road group is estimated by using the local mixture model that is described in detail in Chapter 2. Benchmarking study is highlighted with the comparison of deterioration curves for groups of pavemen sections. Empirical application is conducted on the surveyed data of the national road system with 6510 road sections of one kilometer long for each section. Part of prediction results applying Kyoto model is shown in Figure 4.15.

The estimation results of pavement performance curves shows that the deterioration of road pavement in the southern region is faster than that of road pavement in the northern region. Further investigation and analysis are required to identify the causes of such gap. However, it could be explained by the effects of soft ground condition in the southern area or the impact of flooding in low land region including high ground water level.

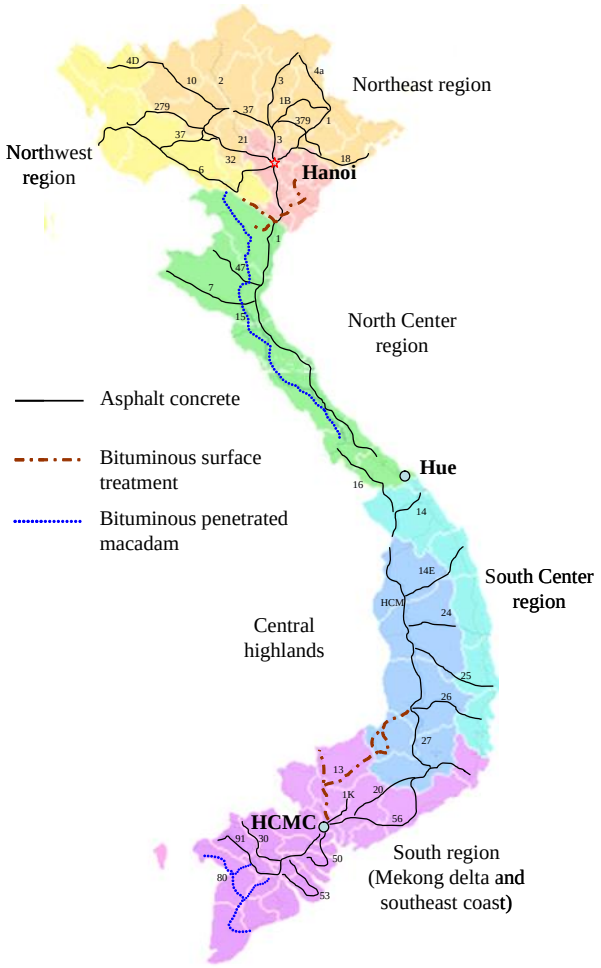


Figure 4.14 Grouping road pavement sections by regions

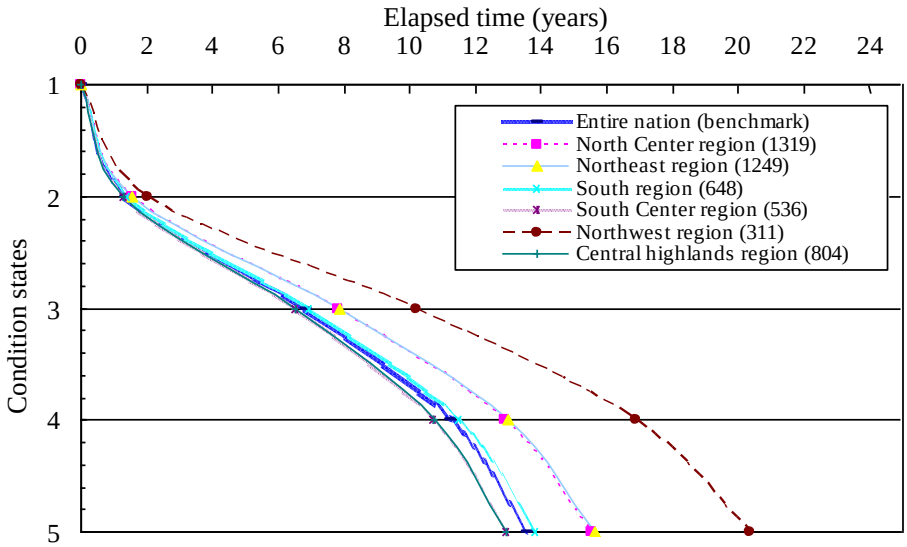


Figure 4.15 Deterioration curves of pavements in different regions

As shown in Figure 4.8, in order to optimize road maintenance, selection of pavement candidate repair sections must be proper based on its priorities. Time-series data supports for clarification the necessary and priority of repair for each section. By applying Benchmarking analysis, heterogeneity factor ε for each group of pavement sections can be determined. Following is the distribution chart of ε for individual group of ten pavement sections of 100m length in each kilometer in National highway No.1 from Km129 to Km150 using pavement condition data collected in 2012 and 2014. Pavement crack ratio collected in 2014 is shown in red line along the road that visualizes pavement conditions. The selection of candidate repair sections becomes quite simple and logical based on pre-set criteria of pavement repair. By showing both current pavement condition in crack ratio and deterioration forecasting result of heterogeneity factor ε by benchmarking analysis on one chart, two types of candidate repair sections can be proposed: (i) sections exceeding criteria as shown in region 1, 2, 3, and 4 in the table below; (ii) sections in fast deterioration with high value of ε. The result in Figure 4.16 for the cases of three 100m long-sections in NH.1 (Km135+500m, Km137+900m, and Km141+300m) shows the similar concern as illustrated in Figure 4.8. Crack ratios in these three sections are quite similar and reaches to the critical value. Benchmarking analysis points out that the third section belongs to region 5 is the most critical section that must be set the highest priority of repair due to its faster deterioration characterized by higher value of ε. Moreover, intensive monitoring for the other sections in region 5 is also strongly recommended.

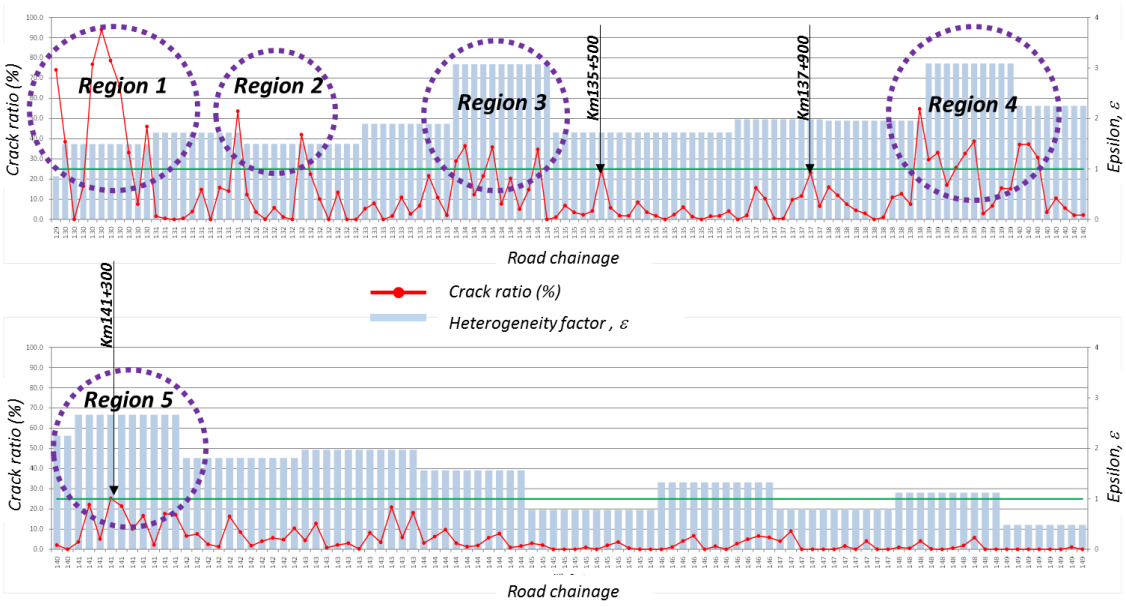


Figure 4.16 Pavement crack ratio and distribution of heterogeneity factor along the road

The difference of deterioration among all groups of pavement sections as the result of benchmarking analysis is visualized in performance curves in Figure 4.17. The fast deterioration of some pavement sections takes negative effect on reduction of pavement life expectancy. In order to achieve requirements on level of service under the constraint of life cycle cost minimization, priority for repair or improvement must be set for these sections in critical speed of deterioration.

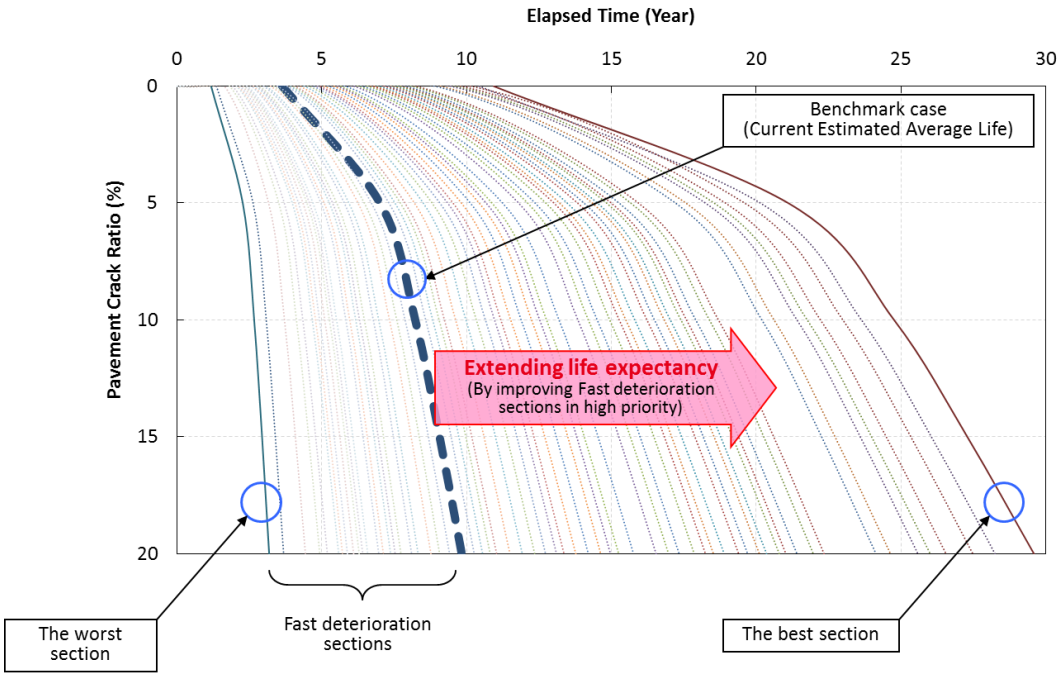


Figure 4.17 Pavement deteriorations curves for groups of pavement sections in NH.1

Regarding to implementation of road management and maintenance within PDCA cycle, there exist interaction among components of the cycle. By application of Benchmarking analysis in "Plan", critical sections with faster and highly accelerated deterioration can be identified that must be selected as candidate sections for further intensive investigation to find out the causes of failures and solutions of proper treatment. Such outcomes from "Plan" support to improve practices of "Check". Pavement condition inspections or surveys should be classified into different categories. The more intensive inspections like structural investigation that are usually costly and time-consuming are applied just for selected sections instead of all sections in case of lacking Benchmarking analysis.

Moreover, it is also possible to improve pavement performance or lengthen its service life by post-evaluation using the latest inspection data such as regular condition surveys. The data as the result of "Check" also support to verify various measures in "Do" that consist of both technical issue in pavement maintenance like materials, technologies, construction, and institutional issues in pavement management such as: type of maintenance contract, demarcation the authority and responsibilities among relevant stakeholders, vehicle weight control, etc,.

4.4.3.4 Evaluation of Policies on Road Management and Maintenance

A road infrastructure is a national asset to be taken over to the next generations, so that road functions need to be carefully maintained for a long duration of road maintenance. The basic principle of road maintenance is to select "right works", "right places" and "right timings" of maintenance and repair works in order to ensure the best economy over the long course of road maintenance. With this, road operators are encouraged to shift from *ex post fact* maintenance to strategic planned maintenance and there must be strategic decisions and policies for both managing the existing road infrastructure and new development to expand the road network including reconstruction heavy deteriorated pavements. In order to make proper decision, supporting tools and informative data must be indispensable. Through implementation road operation and maintenance, these made decisions and policies can be verified and reviewed for modification, renewing if any or just keep applying. The scheme of continual improvement is also emphasized in ISO55000. Relationship between key elements of an asset management system in ISO55000 is simplified in PDCA cycle for formulation framework of implementation (Kawano, H., 2013) [22] including the necessary for post-evaluation to renew policies on road management and maintenance.

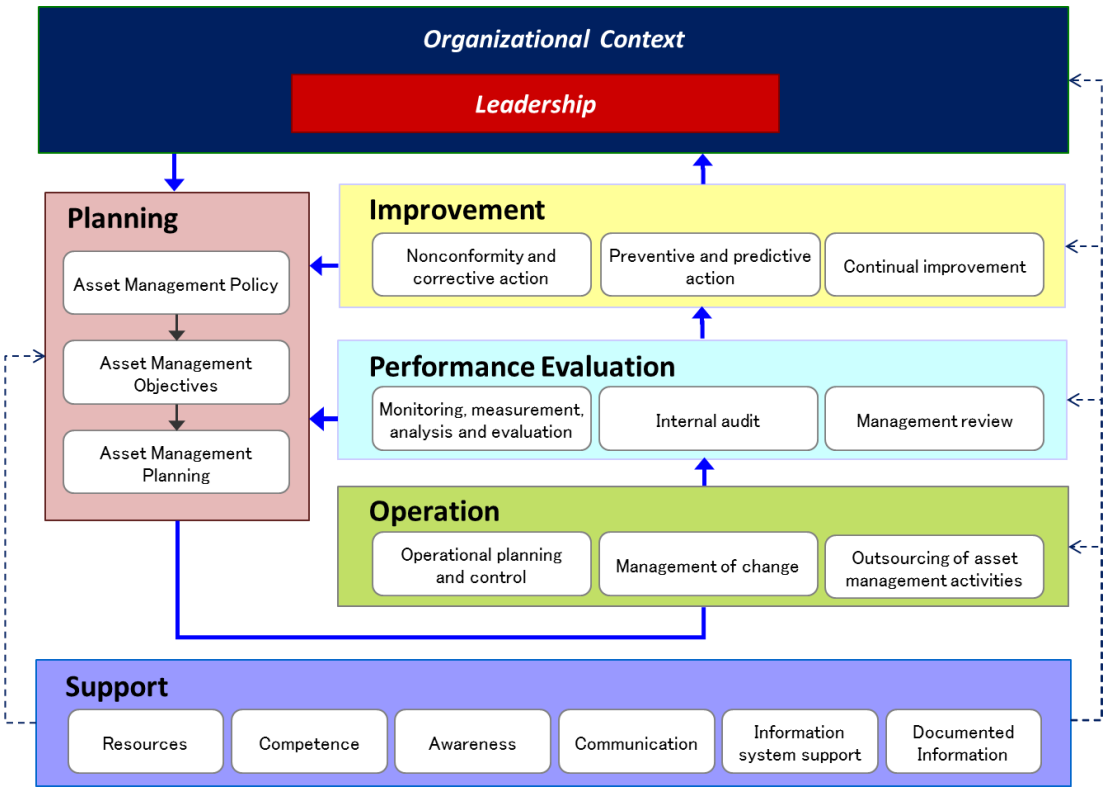


Figure 4.18 Significance of post-evaluation in ISO55000

For each single road asset, there are four main activities in its life cycle that initiates from planning and follows by design, construction and operation & maintenance. For common understanding, the total time of the three first phases is much shorter than the maintenance phase during asset operation. However, in the short time of construction, the budget demand for realizing the road asset in reality is quite big portion in comparison with the total cost or its life cycle cost. And in general, there are many decisions should be made by road administrators and engineers during the implementation of the four main activities. Haas et al,. (1994) [23] introduced the influence level concept, which clearly demonstrates that decision taken during construction phase would greatly influence the cost of pavement maintenance and rehabilitation. Any wrong calculation and improper application of pavement technologies such as pavement type, material selection, construction method may lead to the serious problem of reduction pavement service life. Figure 4.19 also points out that the more earlier approach to systematic maintenance, the better outcomes.

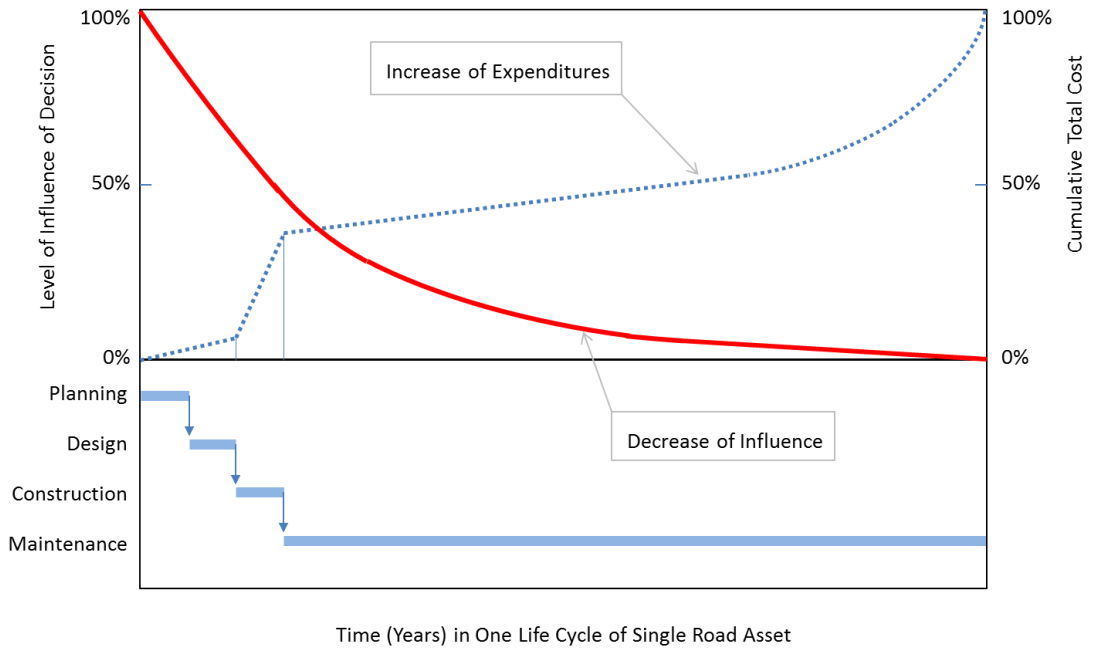


Figure 4.19 Influence levels of decision on Road Asset during its life cycle

In more broader understanding, it should be recognized that effort in maintenance phase is vital for single road asset in optimizing the management and maintenance in the remaining time of its life cycle. In more deeper meaning, any finding in the forth phase of management and maintenance of single road pavement section should be significant feedback for the first three phases for its next life cycles or for planning, designing and construction of many other pavement sections. By maximizing data utilization and analysis, data oriented system of Kyoto model provides best tool for road administrators and engineers to make right decisions.

As description in Section 4.4.2.3, benchmarking analysis in Kyoto model is the essential function for post-evaluation of policies on road management and maintenance.

Regarding road maintenance planning at network level, budget simulation for long-term and mid-term is the function of PMS system to analysis various scenarios of maintenance to find out the best one for implementation. In general, road maintenance and management can be classified into three main scenarios for budget simulation as follows:

- **Scenario-1**: Budget constraint scenario. Among three scenarios, this is the worst case of maintenance in term of budget allocation with the common policy of keeping the current budget level. Under such constraint, the resources allocated for road maintenance is insufficient in comparison with the needs or accelerated deterioration of the roads in general. As the consequence, the exists certain amount of deteriorated road sections in need for repairing but be remained unrepaired due to the shortage of budget. Risk rate is defined to describe the situation of unrepaired remaining as follow:

$$Risk\ Rate = \frac{\sum_{i=1}^{n} l_i}{L} \qquad (4.6)$$

where,

n: number of deteriorated road sections in need for repairing but be remained unrepaired

li: length of road section i^{th}

L: total length of roads in analysis.

- **Scenario-2**: Management level scenario. This scenario is quite common with the positive management and maintenance policy. In this case, maintenance must be conducted to secure a certain level of service (LOS) of the road network. Under the fact of road accelerated deterioration, remaining current level of pavement conditions for the road network to retard its deterioration progress is the common maintenance strategy in this case. Requirement on budget allocation should be higher in comparison with the first scenarios to control the risk of deterioration.
- **Scenario-3**: No budget-constraint scenario. This is the best case of maintenance policy to accept all needs of repair under constraint of a certain targeted LOS for pavement conditions of the whole road network. Usually, budget allocation for this case is very huge to secure for such high LOS.

Figure 4.20, Figure 4.21, Figure 4.22, and Figure 4.23 show outputs of budget simulation in Kyoto model for the case study of national road network in the northern area in Vietnam that consist of 21 routes with total length of 2,303 km. Taking consideration these outcomes including pros and cons in each option, right policy and strategy of road management and maintenance shall be decided systematic and logically instead of political judgement. Such approach also increases transparence and commitment from both government agencies on securing budget allocation and road administrators on provision of corresponding LOS to road users and public.

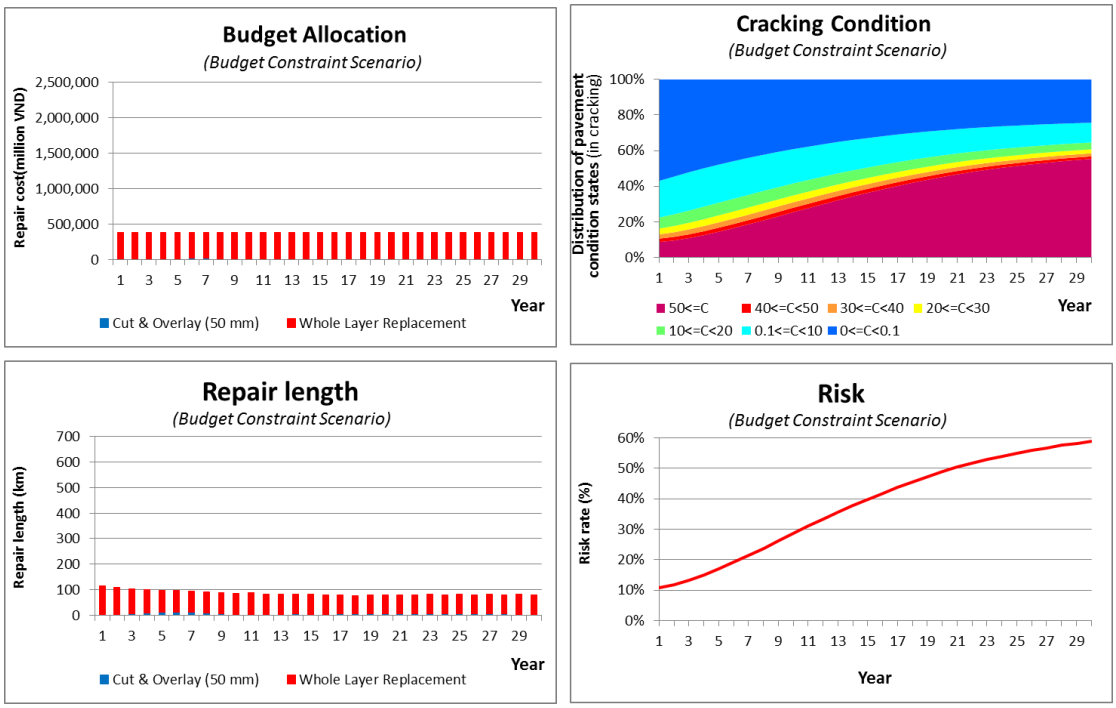


Figure 4.20 Main outcome of budget simulation in Kyoto model - Maintenance scenario 1

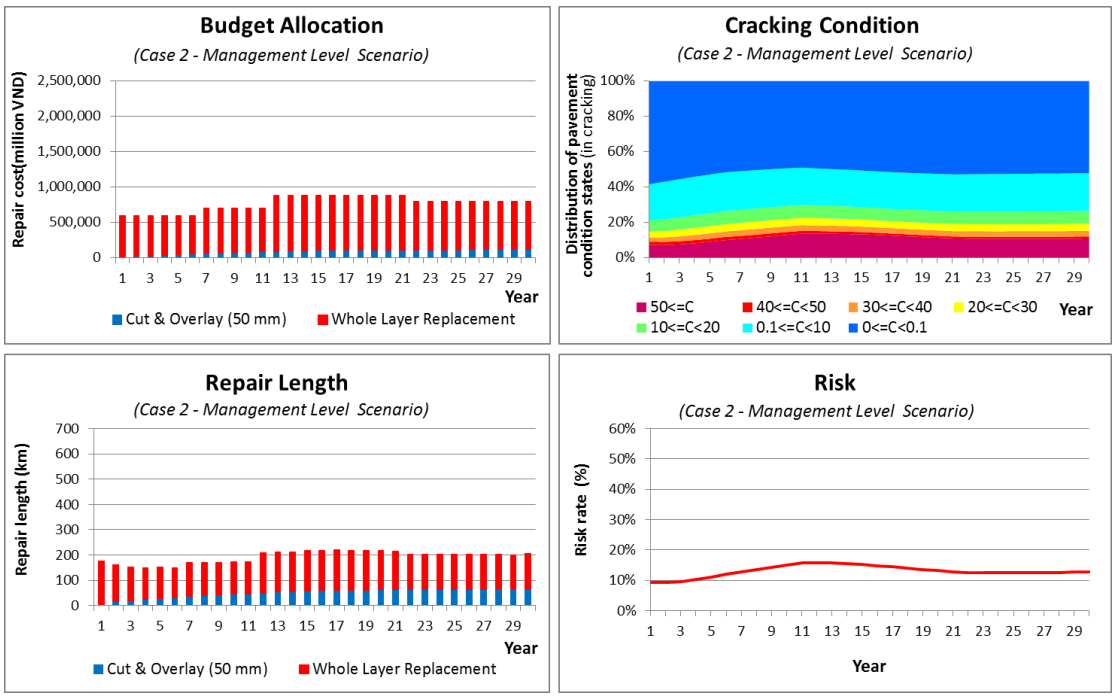


Figure 4.21 Main outcome of budget simulation in Kyoto model - Maintenance scenario 2

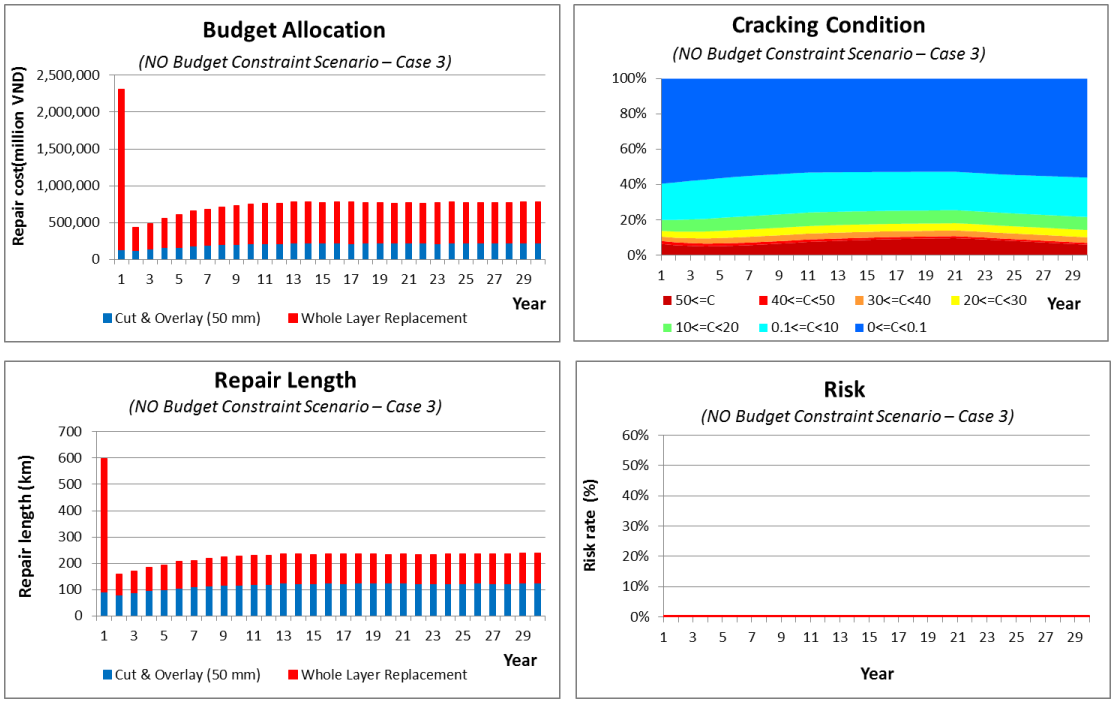


Figure 4.22 Main outcome of budget simulation in Kyoto model - Maintenance scenario 3

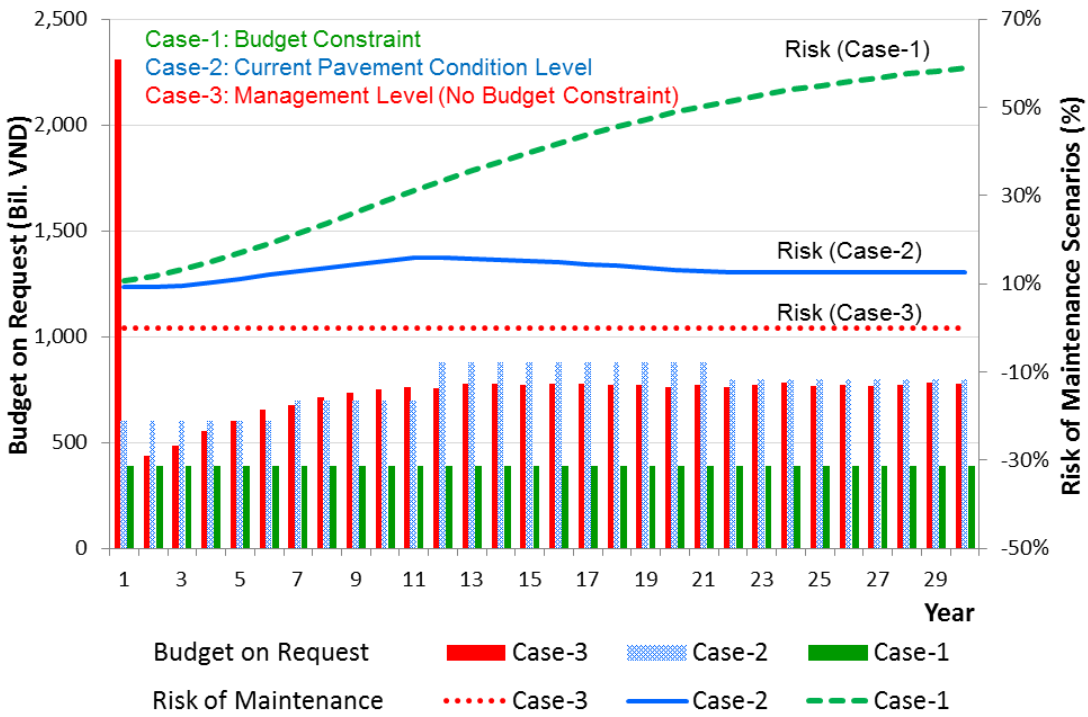


Figure 4.23 Main outcome of budget simulation in Kyoto model - Comparison of maintenance scenarios

4.5 SUMMARY AND RECOMMENDATION

Implementation of PDCA cycle is so significant because any relevant activities can be optimized to improve pavement management and maintenance continuously that may increase pavement longevity, minimize its LCC, and strengthen accountability to road users, tax payers and public. In the PDCA cycle, post-evaluation function in the PMS Kyoto model based on accumulated data from "Check" would bring useful information and valuable recommendations to take action ("Act") for enhancement of "Do" or "Plan" as well. Over the time, PDCA cycle would be enhanced also.

Mathematical models are used to develop the PMS Kyoto model with the logical methodology that is explainable for deeply understanding. Being master of the system, any requirement on customization the model to fit to the actual conditions of application can be satisfied. Based on the platform of Kyoto model, it is possible for road administrators in each region, nation to develop their own customized PMS system to support for their PDCA cycle in their infrastructure asset management.

Within the institutional framework under each application condition, functions of PMS system can be different that may vary from simple functions to sophisticated ones. However, in any case, data works must be the first priority for implementation of PMS in particular and PDCA in general. Infrastructure asset management, time-series data should be one of the valuable asset as defined by the international standard for asset management known as ISO55000. And Kyoto model is planned to support and promote for any minor effort of data works.

As the master of fact, beside the technical efforts, there must be proper institutional arrangement to define the relevant stakeholders of PMS system and the demarcation among their responsibilities from data preparation, system analysis to system upgrading and expansion. It is also expected that the successful cases of Kyoto model implementation shall be expanded to many nations and countries especially in Asian region.

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CHAPTER 5

Practical Process for Development a new Pavement Management System in Vietnam

5.1 BACKGROUND

Road transport in Vietnam shares substantial part of inter-city land transportation compare with other transport means such as railway and inland waterways. Improvement of the national road network is essential and eminent task of the government to enhance development of national economy.

While maintenance burden of national road is increasing yearly due to sharp increase of traffic, rapid expansion of road network and accumulation of dilapidated or old age facilities, annual budget allocated for maintenance of road assets is nearly one half of actual maintenance needs. Hence, establishment of effective road maintenance system that can draw efficient maintenance plan and implementation optimal scheme under the tied budget constraints is direly needed by road agencies. Current problem of accelerated deterioration of road infrastructure also demands for setting high priority for proper actions of maintenance to avoid increasing burden of financial requirement for replacement fully deteriorated sections in the coming time [1](VITRANSS 2).

Various donors including World Bank (WB) and Asian Development Bank (ADB) have been carrying out numerous technical assistances to road agencies including introduction of Pavement Management Systems such as RoSy System and HDM-4. However, roles and responsibilities of donor agencies were not clearly defined and demarcated [2], [3] (SAPI2-2009); therefore, consistent technical assistance to the central road administrator (VRA) was not extended. This entails presence of non-working PMSs in the VRA for years after long application that makes the hard situation in road maintenance becomes more critical especially in budget preparation due to lacking of maintenance goals, budget proposals for both long-term and middle term.

Cognizant to the above situation, it is necessary for development a new PMS to customize and make it adapt to the local conditions in Vietnam with the goal of PDCA cycle implementation for improvement of road management and maintenance.

5.2 CURRENT SITUATION AND PRACTICES OF NATIONAL ROAD MAINTENANCE

5.2.1 Road Classification

According to Decree No.11/2010/ND-CP [4] on February 24, 2010 which was enacted pursuant to the Law on Road Traffic (No.23/2008/QH12) [5], the road system in Vietnam consists of six categories of national road, provincial road, district road, commune road, urban road, and exclusive. Exclusive roads are special roads that connect to industrial zones, military zones, forests, etc. Expressway is quite new in Vietnam with some sections are in operation, and some routes are under construction. The decree also prescribes agencies responsible for the construction and maintenance of the roads as shown in Table 5.1.

Table 5.1 Administrative Road Classification

Classification	**Definition of Road Functions**	**Responsible Agency**	**Total Length** km (*)
National Road	The main axial roads of the nationwide land road network, which are of particularly important effect in service of national or regional socio-economic development, defense and security, including: ▪ Roads linking Hanoi capital with the centrally-run cities; and with administrative centers of the provinces; ▪ Roads linking administrative centers of three or more provinces or centrally-run cities; ▪ Roads linking international seaports, aerodromes with international border gates and main land border gates.	Ministry of Transportation	17,646
Provincial Road	Axial roads within one province or two provinces, including roads linking a province's administrative center with districts' administrative centers or with adjacent provinces' administrative centers; roads linking national roads with districts' administrative centers.	Provincial People's Committee (DOT)	25,434
District Road	Roads link districts' administrative centers with the administrative centers of communes or commune clusters or with adjacent districts' administrative centers; roads linking provincial roads with administrative centers of communes or centers of commune clusters.	District People's Committee	50,603
Commune Road	Roads linking the communes' administrative centers with hamlets and villages, or roads linking communes together.	Commune People's Committee	173,752
Urban Road	Roads lying within the administrative boundaries of inner cities or urban centers.	Provincial People's Committee	18,867
Exclusive Road	Roads used exclusively for transport and communication by one or a number of agencies, enterprises and/or individuals.	(Investor)	9,278
Total			295,580

Note) (*) Statistic data in 2010

Source: Decree No.11/2010/ ND-CP, Law on Road Traffic No.23/2008/QH12

5.2.2 Road Infrastructure System

5.2.2.1 Road Network

With typical land features of being a one-dimensional, stretched-out country from north to south in S-shape with land borders with China, Laos, and Cambodia, transportation plays a key role in economic and social development, as well as national security in Vietnam. The country has a diversified transportation system that consists of different major models: roadways, railways, airways, seaways and inland waterways. Road transportation is the most significant model with the highest contribution in terms of domestic transportation sharing with 2011 million passengers accounting for nearly 92% of the total and 585 million tons of goods accounting for nearly 73% of the whole transport sector according to statistics from fiscal year 2010 [7] (DRVN-2012).

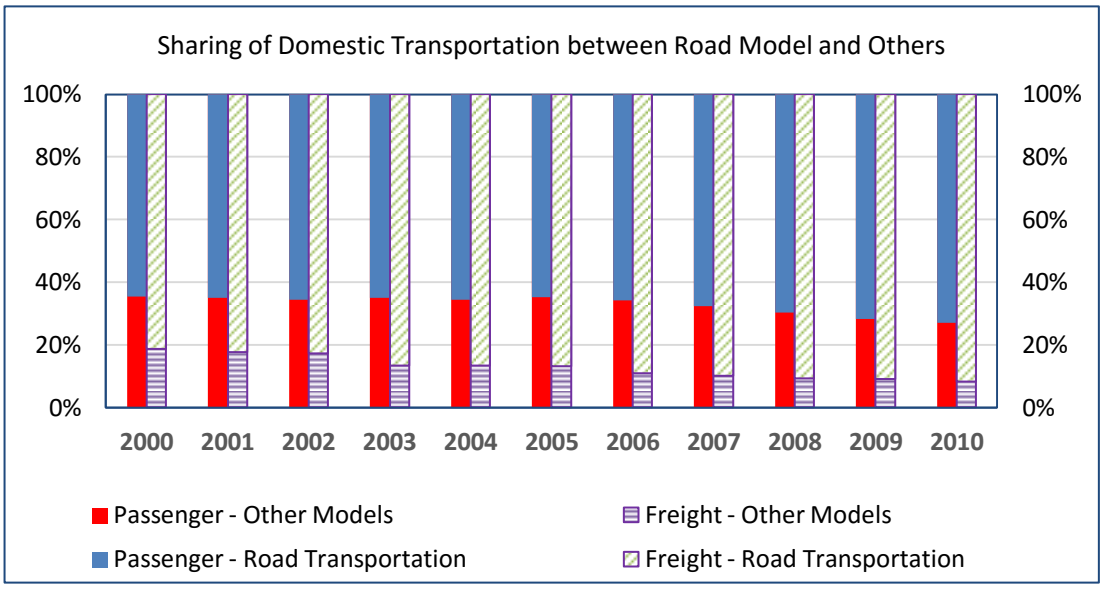


Figure 5.1 Sharing of Domestic Transportation between Road Model and others

Table 5.2 Sharing of Road Sector in Transportation Market

Proportion of road transportation in comparison with the whole transport sector	Statistic data in Year										
	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Goods (%)	64.6	65	65.7	64.9	65.7	64.8	65.9	67.6	69.8	71.8	72.9
Good (Million tons)	145	164	192	225	265	298	339	403	456	514	585
Passengers (%)	81.3	82.4	82.8	86.6	86.6	86.9	89.1	89.9	90.8	91	91.7
Million passengers	621	677	728	931	1042	1173	1332	1473	1629	1761	2011

To cope with such huge demand, road infrastructure has been developed and expanded continuously, not only in quantity with the number of routes, facilities, and road length, but also in quality to conform with increasing transportation demands, especially since Doi Moi or economic reforms. The statistics in the period from 2000 to 2010 showed that the trend keeps continuously increasing that also makes the burden on road infrastructure be harder.

According to the statistics as of 31st December, 2010 [7](DRVN-2012), the total length of road network in Vietnam is 295,580km as shown in Table 5.3, of which 173,752km (58.78%) commune roads, 50,603km (17.12%) district roads, and 25,434km (8.6%) provincial roads, all belong to local road network that consisting of 84.51% in length of the total. On the other hand, the national roads whose total length is 17,646km (5.97%) in 92 different routes in combination with expressways play key function of the primary arterials for the whole nation. Of these, expressway is a very new system that has just been initiated, with some available sections of around several hundred kilometers in operation.

Overall road network has grown at a rate of 10,065 km, by 6%, per year over the past thirteen years since 1997. The national road network forms two north-south corridors, coastal and upland, with east-west roads along the central part of Vietnam. In the north, the national roads form a radial circumferential pattern. In the south, the national road network forms a grid pattern. The coverage of the national roads seems adequate. However, due to the topographic conditions of Vietnam, 39% of the national road network is in mountainous terrain. Therefore, the design standards of nearly half of the national roads are constrained. It also creates problems for road maintenance and is vulnerable to natural disasters, such as landslides that occur with high frequency in the northern region.

Table 5.3 Road Length for the Road Network in Vietnam

Unit: km

Year	Total length	National Road	Provincial Road	District Road	Commune Road	Urban Road	Exclusive Road
1997	164,620	15,071	1,653	32,907	89,372	5,213	5,524
1997	164,740	15,071	16,653	32,907	89,372	5,213	5,524
1998	171,071	15,286	17,097	34,519	92,558	5,534	6,077
1999	180,950	15,392	17,653	35,509	99,913	5,755	6,728
2000	183,177	15,436	18,344	36,840	99,670	5,919	6,968
2001	201,558	15,613	18,997	37,013	117,017	5,921	6,997
2002	201,295	15,824	19,916	37,947	114,643	5,944	7,021
2003	216,790	16,118	21,417	46,508	118,589	8,264	5,894
2004	223,287	17,295	21,762	45,013	124,942	6,654	7,621
2005	230,503	17,295	23,990	47,109	126,869	7,808	7,432
2006	268,777	16,125	24,822	50,844	155,968	15,182	5,836
2007	271,535	17,339	23,905	54,181	158,965	10,075	7,070
2008	277,560	16,913	24,750	43,520	175,329	9,558	7,490
2009	279,927	16,758	25,449	51,721	161,136	17,025	7,838
2010	295,580	17,646	25,434	50,603	173,752	18,867	9,278

Source: DRVN (December 2010)

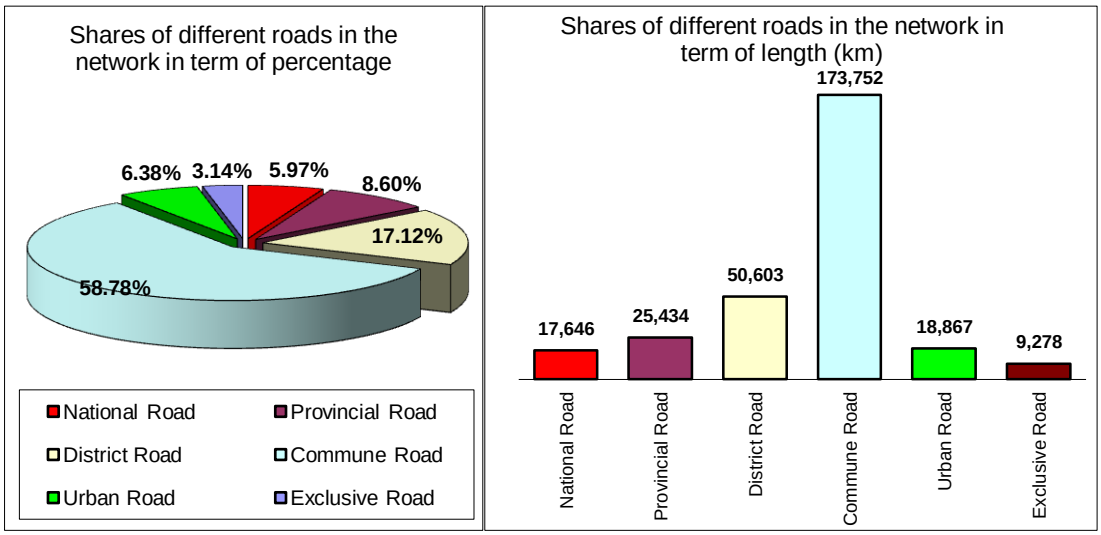
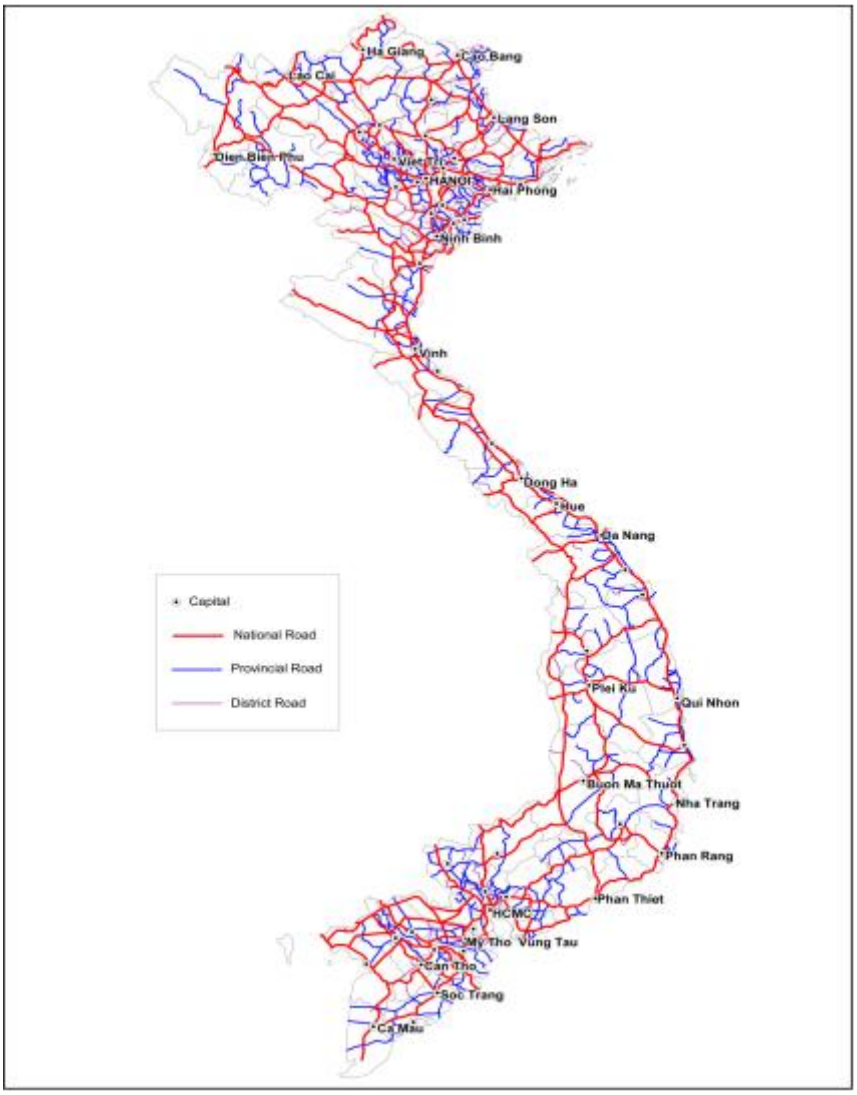


Figure 5.2 Share of Roads in the Network (Statistic data 2010)



Source: Directorate for Roads of Vietnam

Figure 5.3 Distribution and Location of National Road Network in the Mainland of Vietnam

Among developing countries, Vietnam is a good example of proper strategic policies to promote industrialization, modernization and national development. Many great achievements have been obtained in different fields including the transport sector. It must be confirmed that the fruits of road infrastructure development at present have been achieved thanks to the right direction and proper governing of the Party and the State, domestic efforts, and great support from donors, international organizations, and many countries including Japan.

Beside the above administrative classification of roads, there is another technical classification to classify roads into six classes in terms or design speeds regulated in national standard of highway design TCVN 4054: 2005 as shown in following table.

Table 5.4 Road Technical Classification in TCVN 4054: 2005 [8]

Road Class	**I**	**II**	**III**		**IV**		**V**		**VI**	
Terrain (*)	F	F	F	M	F	M	F	M	F	M
Design Speed (km/h)	120	100	80	60	60	40	40	30	30	20

(*). F: Flat and hilly terrain; M: Mountainous terrain

5.2.2.2 Pavement Conditions

In common understanding, pavement is the main road facility that takes great contribution for securing level of services in both loading capacity and its performance in riding comfort, skid resistance, deformation resistance and so forth. Rigid and flexible pavements are two main categories. While cement concrete is the typical pavement of the first category, the second one can be classified into various types as shown in Table 5.5.

The pavement ration of asphalt concrete and bituminous pavement constitutes 31 percent of all road networks. Regarding to major roads, same pavement ratio goes up to 94 percent for national roads, followed by 76 percent for provincial roads, 35 percent for district roads and 73 percent for urban roads. A major pavement type, asphalt concrete pavements occupy 64 and 47 percent for national roads and urban roads respectively, bituminous pavements 59 and 29 percent for provincial roads and district road respectively, gravel surface 44 percent for district roads and earth type 86 percent for commune road. This clearly shows a more economical pavement structure has been applied in accordance with road classification.

Moreover, utilization of asphalt concrete becomes more popular for national roads and urban ones that usually has strong requirement on surface performance. In total, application of cement concrete pavement is wider than asphalt concrete pavement in term of road length with the highest share of around 82 percent on commune roads including rural ones due to the

applicability of simple and manual construction method without requirement of heavy machines or specialized equipment as slip-form concrete paver for high-class roads.

Table 5.5 Road Classification by Pavement Type

Unit: km

Road Classification	Total Length	Sub-total Length with Pavement of					
		Asphalt Concrete	Bituminous Pavement	Cement Concrete	Aggregate, Macadam	Compacted Soil	Others
National Road	16,758	10,751	4,999	367	366	0	275
Provincial Road	25,449	4,398	14,904	620	3,027	15	2,485
District Road	51,721	3,039	14,833	5,189	13,974	426	14,260
Commune Road	161,136	2,820	16,516	36,904	33,315	1,978	69,603
Urban Road	17,025	8,063	4,403	1,700	1,601	28	1,230
Special Road	7,838	1,097	744	356	1,554	14	4,073
Total	279,928	30,168	56,399	45,136	53,837	2,462	91,926

Source: Infrastructure & Traffic Safety Department, DRVN, Statistic data in 2009

5.2.3 Road Administration

The principal governing regulation for road transportation is the Law on Road Traffic, which was initially enacted by the National Assembly on June 29, 2001. The law was revised in 2008 and has come into effect since July 1, 2009 (No.23/2008/QH12) that prescribes roads traffic rules, road infrastructure facilities, vehicles in traffic and road users, road transportation and state management of road traffic and road infrastructure including definition for six administrative categories of roads.

Following the law enactment, the government issued Decree No.11/2010/ND-CP dated on February 24, 2010 which prescribes administrative organizations to be managing and operating all road networks in Vietnam. Road network can be divided into two main categories of national road network and local road network for proper demarcation responsibilities of management and maintenance by the central competent authority of the Ministry of Transport (MOT) and the local government authorities Figure 5.4. Owners of special roads must be responsible for the task of management and maintenance. There are many other competent agencies or authorities that involve directly or indirectly in road management and maintenance such as the key role of DRVN and PDOTs in direct tasks for national roads and local road respectively, budget works of the Ministry of Finance (MOF) as well as provincial Department of Finance (DOF), and so forth.

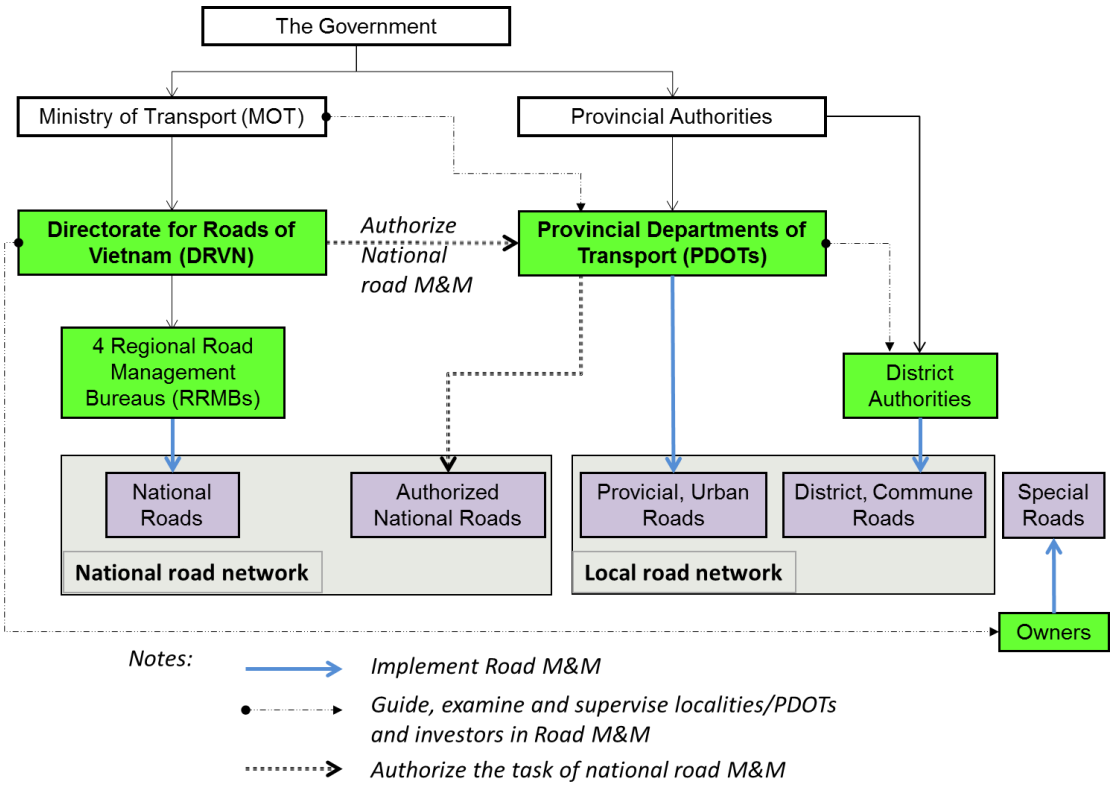
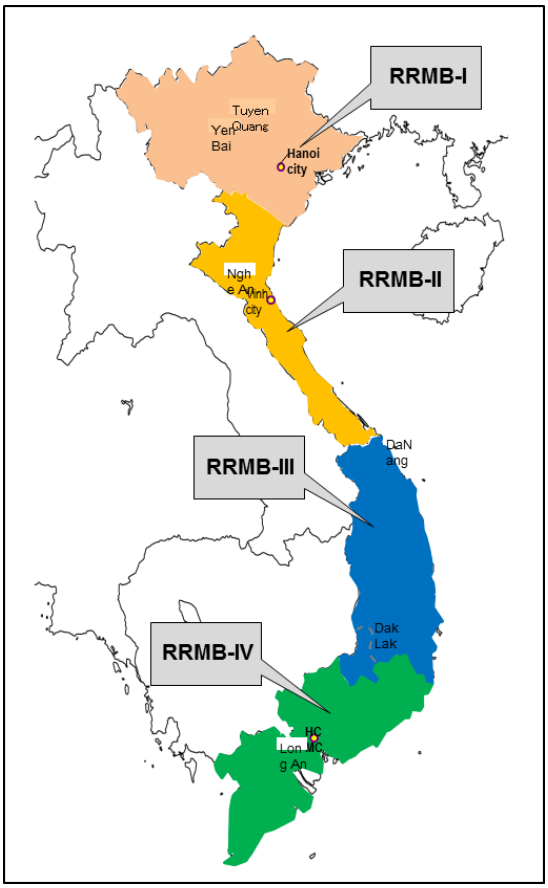


Figure 5.4 Definition of Road Categories

Being one subsidiary organization of MOT, DRVN takes the function of state management and be responsible for road sub-sector in transportation sector that consist of road management and direct maintenance and operation for national road network throughout the country.

Since the beginning of arrangement for road management, typical feature of one-dimensional nation had been seriously taken into account for setting up proper regional road management units. Four units of regional road management (RRMUs) had been established to implement their functions and assignment of national road management and maintenance within their assigned jurisdiction. Since 2013, enhancement the function of state management for regional road management organizations has been emphasized and conducted. Regional road management bureau I, II, III, IV (RRMBs) have been re-structured from four former RRMUs with their jurisdiction as shown in following figure [9] (Decision No. 60 /2013/QD-TTg).



RRMB: Regional Road Management Bureaus

Figure 5.5 Jurisdiction area of Regional Road Administrators (RRMBs)

Due to the wide distribution of national roads throughout the country, part of the national road network with the proportion of around 50 percent has been authorized for many relevant PDOTs for direct management and maintenance. In any case, RRMBs must be in charge for main routes of national road network.

Previously, field works of road management and maintenance had been assigned for road repair and management companies (RRMCs) or provincial road repair and management companies (PRRMCs) by the headquarters of RRMUs or authorized PDOTs respectively. Direct assignment from the regional headquarters to field companies had been applied widely for a long time that can be recognized as the main cause of losing motivation for improvement as well as poor decision making function of maintenance companies. Pursuant to the Law on Enterprise code No.60/2005/QH11 [10] and government policy on privatization of state owned enterprise (SOE), all these RRMCs must be re-structured for privatization that were no longer belong to RRMUs.

During such privatization process, proactive response can be seen in combination with some new approaches of maintenance contract from introduction of WB including pilot projects of performance-based contract (PBC) and Japanese practices on road administrative system

[11] (Kanoshima, 2012), the biggest change of road administrative system in Vietnam was made that is stated in the Prime Minister's Decision No. 60/2013/QD-TTg dated on October 21, 2013 on stipulation functions, duties, authority and organizational mechanism of DRVN [9]. In the decision, state management function of road administrative system is strongly emphasized even for field units with the goal for capacity enhancement in road management and maintenance for governmental organizations that can handle current and coming challenges in this field. All maintenance and repair works are outsourced to contractors that are totally out of road administrative system (Figure 5.6). In the road administrative system, headquarters focus on management tasks including authorization to PDOTs and supervision their implementation of works on authorized national roads, and contract work with outsourcing contractors that require for two main tasks for field force of sub-bureaus (SBs) in field management of road infrastructure and supervision maintenance and repair works of contractors.

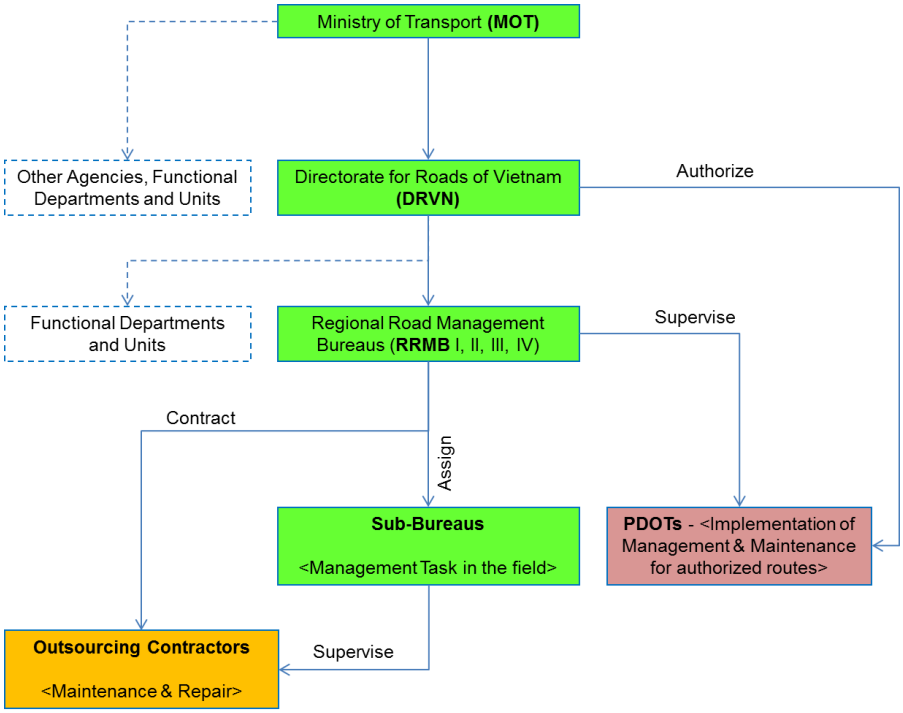
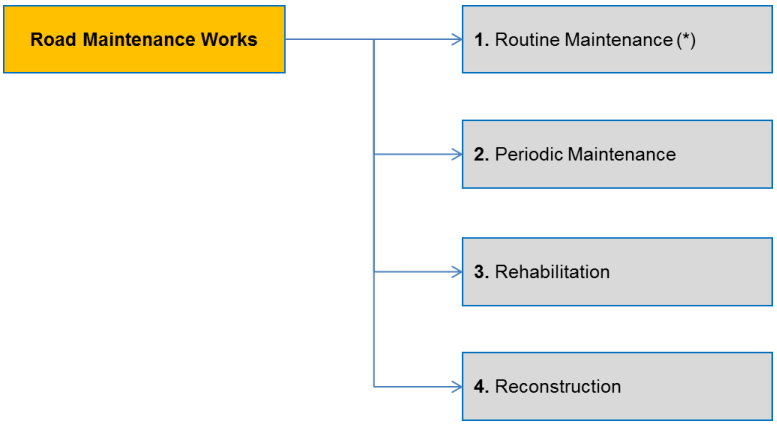


Figure 5.6 Flow of Work Assignment or Agreement of National Road Management and Maintenance

5.2.4 Practices of Road Management and Maintenance

5.2.4.1 Classification of Road Maintenance

The basic concept for road maintenance is summarized here in this section, including definitions, the outlines of activities, the general frameworks and the functional requirements for road maintenance.



(Note)

(*) Unscheduled maintenance which includes the restoration works for disasters and traffic accidents is categorized in the "Routine Maintenance" or reactive maintenance.

Figure 5.7 Structure of Road Maintenance Works

In general, maintenance work is defined and classified based on its objectives. Figure 5.7 shows the structure of road maintenance works, and also Table 5.6 shows the objectives of each activity, followed by the brief explanation. Four maintenance activities are in general incorporated in the long duration of road operation with different objectives.

Table 5.6 Maintenance Works and Objectives

Road Maintenance Works	Objectives				Typical Activities
	Maintain Serviceability	Retard Aging	Maintain Strength	Increase capacity	
Routine maintenance	⊙				Cleaning of facilities Small repair works (Reactive maintenance) Temporary restoration from the damaged of disasters or accidents etc.
Periodic Maintenance	⊙	⊙			Medium/big repair works (Proactive maintenance)
Rehabilitation	⊙	⊙	⊙		Replacement of facilities without functional upgrade
Reconstruction	⊙	⊙	⊙	⊙	Reconstruction of facilities with functional upgrading

(1) Routine Maintenance

Routine maintenance has an objective to maintain serviceability including safety of the road facilities. It is promptly implemented in response to the development of deficiencies that negatively impact the safe and efficient operation of the facility.

Main activities categorized in the routine maintenance include patrolling, the cleaning of facilities, small repair works and disaster and accident restoration works, so that the quick detection and the early repair of failures are the key for success of this maintenance.

Small repair works are categorized as ex-post fact maintenance works which are reactive and not proactive to the failures, and must be conducted aiming to restore a facility to an acceptable level of service without expectation of taking remarkable effect on the facility service life, the strength and its functions as well.

Unscheduled works are those aiming to return facilities back to a minimum level of service, while permanent full-scale restoration is generally carried out later on in the rehabilitation or in the reconstruction scheme.

(2) Periodic Maintenance

Periodic maintenance is a planned strategy of cost-effective treatments to existing road facilities with objectives of preserving the facilities, retarding structure deterioration, extending facility life-cycle, thereby maintaining or improving the functional conditions of the facilities without significantly increasing the structural capacity.

Periodic maintenance is a key maintenance activity that can retard the aging of road facilities until the time of expensive rehabilitation, thereby contributing to reducing life cycle cost for road facilities. However, it requires strategic and proactive measures in planning and in implementation.

Main activities consist of medium-scale or big-scale repair works to cope with road facility deterioration and to reinforce structures, which generally require large spending on the works. In order to ensure efficiency in planning and in implementation, a mid-term or a long-term maintenance plan is generally incorporated as a base of planning in order to find out repair locations, repair works and repair timings in the long course of maintenance. Also, these maintenance plans are expected to contribute to stabilization of maintenance budget and investment in the course of maintenance.

Periodic maintenance also covers preventive action that is taken to prevent occurrence of pavement failure and to preserve its function whereas corrective action is taken to prevent its distress recurrence (ISO 55000) [12]. Basically, preventive action is carried out while pavement is functionally available and operable or prior to the initiation of functional failure that also requires for sufficient budget allocation, one big challenge in Vietnam. Thus, corrective action becomes most popular.

(3) Rehabilitation

Rehabilitation is to restore the strength of facilities by replacing a large part of road facilities with the equivalent structure without functional upgrading in general, thereby maintaining the strength of facilities. Rehabilitation is applied to the facilities that have failed in function or become functionally obsolete.

(4) Reconstruction

Reconstruction aims to upgrade road facilities in conformity with the latest technical standards and to make additional functions to the original facilities like adding two more lanes on a roadway as an example.

General framework for maintenance works including missions, responsibilities, work types and functional requirements for each activity is shown in Table 5.7. The functional requirements explain the functions that each activity is equipped with in order to fulfill missions and responsibilities as well.

Table 5.7 General Framework of Road Facility Maintenance

Functional Requirements	1. Routine Maintenance	2. Periodic Maintenance	3. Rehabilitation	4. Reconstruction
1. Missions and Responsibilities	• Quick detection of incidents • Quick countermeasures to the incidents	Planned investment by asset management aiming to minimize facility life-cycle costs in the course of maintenance	Maintain strength of facilities	Upgrade functions of facilities
2. Work Types	• Daily Patrolling • Cleaning of facilities • Small repair works • Restoration works (Temporary) for the damages given by disasters and accidents	• Periodic inspection and data preservation • Planning mid-term and long-term maintenance plans • Medium/big repair works	• Detail survey and design by consultants • Construction project management and quality management, same as those for construction projects	• Detail survey and design by consultants • Construction project management and quality management, same as those for construction projects
3. Functional Requirements	Functions specialized for the routine maintenance are required. (1) Legal framework (2) Organization and staff (3) Project formulation and approval (4) Design standards (5) Cost estimate manual (6) Budget proposal and approval (7) Contract management (8) Standards for work and quality management	Functions specialized for the periodic maintenance are required. (1) Legal framework (2) Institutional arrangement (3) Project formulation and approval (4) Design standards (5) Cost estimate manual (6) Budget proposal and approval (7) Contract management (8) Standards for work and quality management	Same functions as those for construction projects are required.	Same functions as those for construction projects are required.

In Vietnam, classification of national road maintenance works is stipulated in the Circular N0.10/2010/TT-BGTVT by Transport Minister [13]. The road maintenance works fall into

three types: (i) routine maintenance, (ii) periodical maintenance, and (iii) unscheduled maintenance.

Definition of road maintenance can be different under each set of conditions for the purpose of united implementation in reality. However, such classification is also so meaning in the sense of maintenance optimization by keeping a balance between planned and unplanned maintenance (AASHTO-2011) [14].

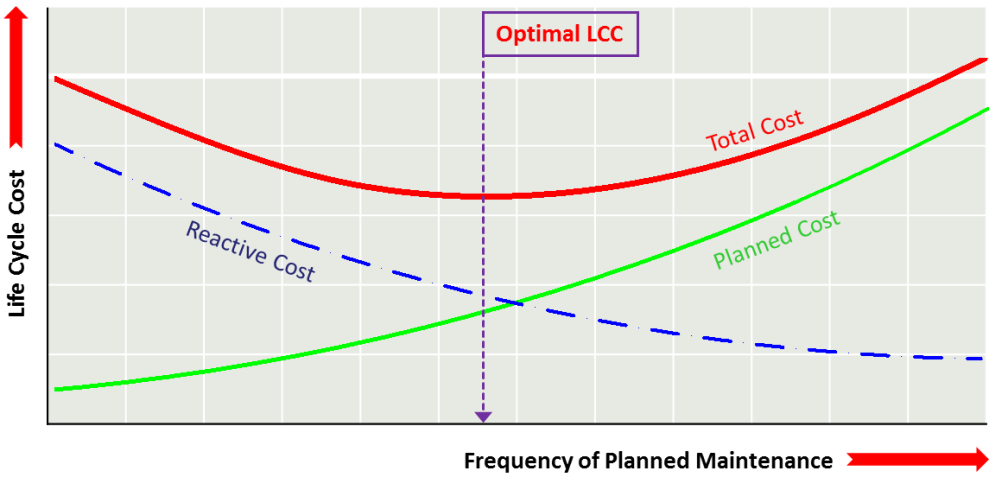


Figure 5.8 Illustration the Principle of Maintenance Optimization

As shown in the Figure 5.8, the higher the frequency of planned maintenance implementation, the much increase of its costs that also leads to the improvement of road level of service and the reduction in probability for reactive or unplanned maintenance. The situation is totally opposite in case of trying to limit implementation of planned maintenance. Therefore, in any case, the principle of maintenance optimization for both planned and unplanned works is strongly recommended even for strong economics to obtain optimal LCC.

Table 5.8 shows stipulation of the Circular 10 on frequencies of periodic pavement maintenance for both medium-scale or big-scale repair works. The medium repair is the repair of breakdowns and structural deterioration, which may possibly affect vehicle traffic and lead to traffic accidents. The big repair indicates the repair works to be conducted upon occurrence of breakdowns or degradations in the various parts of the road structures and to restore them to the original functions and quality. Such regulation can be understood as the framework in average. In implementation, road administrators should customize the regulation in each case for optimization their maintenance works.

Table 5.8 Work Frequencies of Medium and Big Repairs

No.	Road pavement type	Interval in years for	
		Medium repair	Big repair
1	Asphalt concrete	4	12
2	Cement concrete	8	24
3	Bitumen-mixed crushed rocks, black crushed rocks	3	9
4	Bituminous penetrated macadam; Single, Double or Triple bituminous surface treatment	3	6
5	Macadam, Crushed rock aggregate	2	4
6	Natural aggregate	1	3

Source: MOT's Circular No.10 / 2010 / TT-BGTVT (April 19, 2010)

5.2.4.2 Inspection of Road Facilities

Road and bridge inspections have been carried out in accordance with the regulations in technical standard of road routine maintenance 22TCN-306: 2003. It prescribes the inspection procedures including inspection classification, inspection organizations and frequencies of inspections, but little about the details of inspection points and measurements, i.e. how to inspect and where to inspect [15]. A guideline showing information on the inspection points and measurements using simple and informative illustration must be needed.

Table 5.9 Outline of Road Inspection

Road Structures	Names of Inspection	Frequencies	Implementation bodies	Inspection Points
A. Roads & Structures	Routine Inspection	Once a day	Road Patrol, RRMCs	
	Periodic Check	Monthly	RRMCs, Repair Team	Pavement Surface, Drainage system, Road Signals, Dikes, etc.
		Quarterly	RRMU/PDOTS, RRMCs	
	Special Check		RRMUs/PDOTs	Pavement & Sub-grade Strength, Evenness
B. Bridges	Routine Check		Repair team, Technicians	Bridge deck, Beams, Bearings, Abutments, Piers
	Periodic Check	Twice a year: Before and after rainy season	RRMUs/PDOTs	Erosion, scour of piers etc.
	Unscheduled Check	Unscheduled As required	VRA, RRMUs/PDOTs, RRMCs	Bridge defects and damages
	Special Check		RRMUs/PDOTs	Subgrade in soft soil or sliding curb, Strength of Pavement, Bridge
	Bridge Inspection	Initial inspection; to record initial status of structures before traffic operation.	RRMUs/PDOTs	Whole bridge
		Follow up inspections; 10 years later, then 5-7 year intervals	RRMUs/PDOTs	Whole bridge

Source: "Technical Standards on Road Routine Maintenance", May 28, 2003, MOT.

Routine and periodic inspections have been carried out by the staffs belonging to maintenance companies and RRMUs/PDOTs. Of these inspections, the periodic inspection often requires high engineering knowledge and expertise in performing the inspection and in making diagnosis of the structural deterioration. It is very important to incorporate professionalism and objective views into the periodic and the special inspections.

Upon completion of road inspections, diagnosis of road structure deterioration is carried out to make a judgment on the extent of damages, followed by the selection of repair works. However, the diagnosis has been commonly done based on the engineer's experience and judgment due to the lack of appropriate guidelines. There is no guideline available for diagnosis except for the one on pavement structures. Also, under the current budget constraints, the selected works are basically the reactive maintenance works.

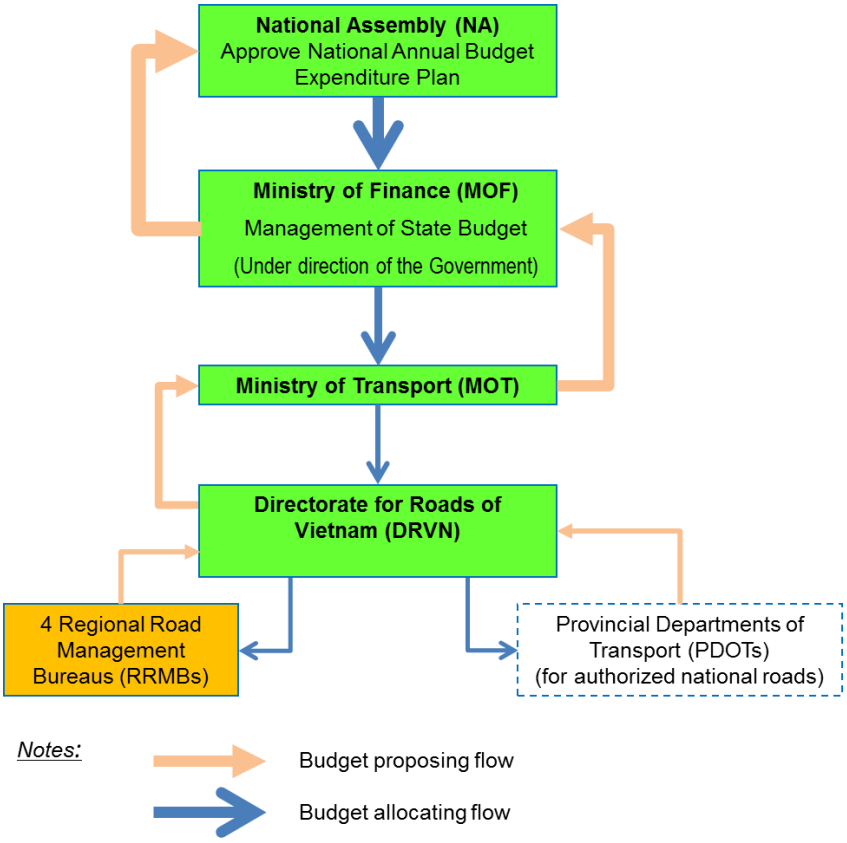
However, with the revision of technical standard of road routine maintenance 22TCN-306: 2003 to formulate the new version code TCCS 07:2013/DRVN, regulation on road facility inspection is separated. It is planned for development a new specialized standard or manual on road facility inspection.

5.2.4.3 Road Maintenance and Budget Planning

There have been no officially approved long-term or middle-term maintenance plans which can indicate strategic investment perspectives on the national road assets due to lacking of proper PMS system and data also. However, several trials of plan formulation focusing on pavement structures have been made since early 2000's upon receiving international donor assistance. On the other hand, an annual plan has been playing a key role in formulation of budget proposal for every fiscal year. Further details of these plans are elaborated below:

Annual road maintenance plan is the prevailing means of formulating a budget plan. Regional agencies, including RRMBs and PDOTs, play a key role in preparation of annual plans. Annual plan includes the budget plan for routine maintenance and periodic maintenance consisting of medium repair and big repair. In formulating annual plans, much effort is directed to planning repair works rather than routine maintenance since works for the routine maintenance are prescribed in the maintenance norms and standards as lump sum calculation.

As shown in the Figure 5.9, the budget proposal for road maintenance originates from the regional organizations of RRMBs and PDOTs. The budget proposal and distribution procedures follow the following steps



Source: Directorate for Roads of Vietnam

Figure 5.9 Budget Proposal and Distribution Flows

(1) Budget Proposal Processes

- In every year, RRMBs and PDOTs prepare budget plans and submit to DRVN.
- DRVN integrates all individual plans into single format, examines the contents and then submits this draft budget proposal to MOT around October.
- Once receiving the proposal from DRVN, MOT examines again the contents and integrates in MOT's annual plan for the whole transport sector to Ministry of Finance, where budget proposals assembled from various ministries and sectors are examined again referring to the expected revenue amounts.
- The final budget proposal is subject to the decision of the National Assembly.

(2) Budget Distribution Processes

- Based on National Assembly's approval on the State budget allocation and Government's direction, MOF will decide to allocate the estimated amount to MOT.
- Then, MOT will order DRVN to draft budget distribution plans to the regional agencies and report to MOT.
- After appraising these draft plans, MOT will make a final decision on the budget allocation to the regional RRMBs and PDOTs for the national road maintenance.

- DRVN convenes a meeting with all RRMBs and PDOTs to reallocate the allocated budget.
- Following the decision made at the meeting, RRMBs and PDOTs re-arrange their initial annual maintenance plans for finalization under allocated budget constraints to DRVN for approval.
- Upon approval, RRMBs and PDOTs move forward to the next step of making maintenance contracts with maintenance contractors.

(3) Current Budget Status

Vietnam has been facing with a chronic shortage of budget for national road maintenance. Table 5.10 shows proposed and allocated budgets for the past ten years, the growth of proposed and allocated budgets for the past ten years is shown in Figure 5.10. It is often said that budget allocation is usually less than half of the budget requirement. In addition, the budget allocation keeps growing gradually in recent years with lower rate in comparison with that one of price escalation of maintenance unit costs in terms of labor and materials or inflation that resulted in reduction in road repair scope and scale.

Due to the budget constraint, RRMBs and PDOTs cannot but focus their resources to the most seriously damaged roads structures or sections with high priority for securing traffic safety and conducting corrective maintenance or repair works. Under such situation, it is quite hard for implementation of preventive maintenance.

Table 5.10 Budget Proposal and Allocation for Road Maintenance

Unit: million VND

Year	Proposed Budget			Allocated Budget		
	Total	Routine Maintenance	Periodical & Unscheduled Repairs	Total	Routine Maintenance	Periodical & Unscheduled Repairs
2002	1,352,067	264,197	1,087,870	661,791	182,680	479,111
2003	1,694,910	311,310	1,383,600	1,382,017	243,990	1,138,027
2004	1,885,155	328,605	1,556,550	1,056,484	284,200	772,284
2005	2,583,809	381,502	2,202,307	1,137,392	326,180	811,212
2006	3,272,701	474,796	2,797,905	1,704,300	433,000	1,271,300
2007	3,400,400	510,060	2,890,340	2,101,992	469,797	1,632,195
2008	2,860,000	690,000	2,170,000	2,080,889	518,892	1,561,997
2009	3,126,400	757,288	2,369,112	2,140,328	546,611	1,593,717
2010	4,424,000	1,028,000	3,396,000	2,380,717	627,089	1,753,628
2011	6,167,980	1,387,796	4,780,185	(*) 3,409,946	767,238	2,642,708
2012	8,904,463	2,270,638	6,633,825	2,671,339	798,730	1,872,609

(Source: DRVN, 2012)

Notes) in fiscal year 2011, addition amount of VND 819 billion from selling the toll collection right at toll plazas Hoang Mai, Ban Thach, Bai Chay and T1 on NH51 that increases allocated budget.

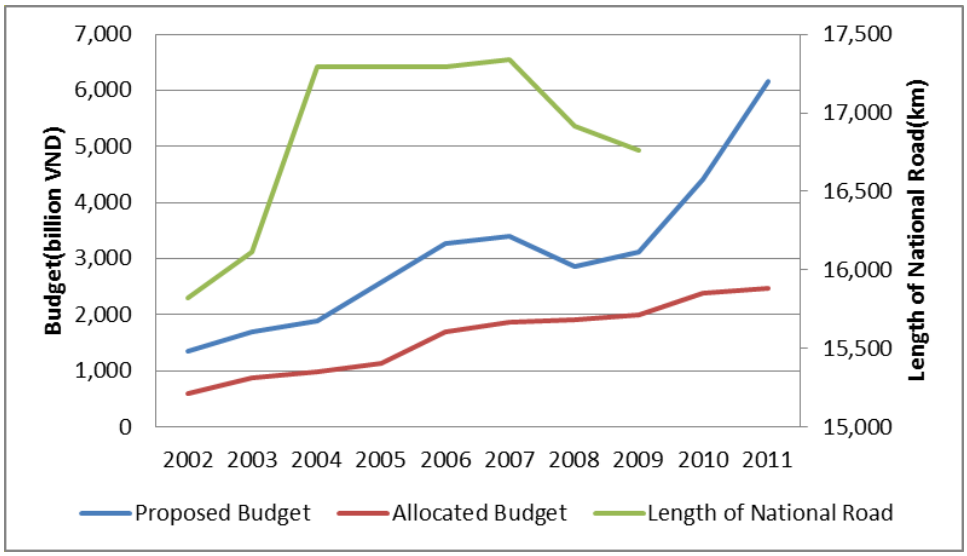


Figure 5.10 Growth of Budget Proposal and Allocation

5.2.5 History of Application of PMS Systems and Lessons Learned

5.2.5.1 Development and deployment of HDM-4

Development of the integrated road investment evaluation model for developing countries by the World Bank became the prototype of HDM back in 1968. The first model was the road cost model (HCM: Highway Cost Model) developed in 1971 by the Massachusetts Institute of Technology (MIT). Afterwards, the Traffic Research Laboratory (TRL) of the United Kingdom executed large-scale field surveying in Kenya in collaboration with the World Bank (WB). The results of the field surveying for empirical study were used to modify the prototype of the road investment model (RTIM). Additionally, this result was used to make an extended version of HCM and RTIM, and became the road design and maintenance management standard model (HDM). In 1994, the microcomputer version HDM-95 (Archonodo-Callao) was made as the result of field surveying and advancing computer technology. Version 2.0 of HDM-4 was released in 2006 through HDM-III, Version 1.0 of HDM-4 in 2000 and Version 1.3 of HDM-4 in 2002 (SAPI2, 2009) [2]. HDM-4 is a software tool that is used to appraise the technical and economic aspects of road investment projects, excluding a database function.

Various versions of the models have been widely used in a number of countries, but success stories in operation have not been reported or released to the public very often. The model has been used to investigate the economic viability of road projects and to optimize economic benefits for road users under different levels of expenditure.

Since the initial introduction in Vietnam for national roads in 1988, six trials were conducted up to 2006 by the WB and the Asian Development Bank (ADB). The HDM-4 trials carried out from 1998 to 2006 mainly used a common dataset format to formulate datasets for analysis,

without relying on an external database. In 2007, the Vietnamese Road Agency (VRA) made a decision to use database software as the official database software in an attempt to convert data into the HDM-4 dataset. This is because HDM-4 can prepare its dataset in two ways: by directly constructing datasets in the HDM-4 format, or by converting and importing data from an external database to formulate datasets for analysis.

5.2.5.2 Development and Deployment of the Database System

In 2007, the VRA made a decision to use the database system as a database tool for national road maintenance management. The first data collection and input to the database system by VRA on their own was carried out in 2007. However, after the field survey, many severe issues with this database system had been pointed out. There were some typical points: (i) insufficient functions to support daily maintenance and management, (ii) difficulties for the regional staff to handle the database and the system due to lacking technical support from the supplier, including limited training, (iii) difficulty in sharing the data inputting job among staff, (iv) insufficient functions for data input control; and (v) less practical reporting functions.

The database system was finally expected to play a role as an external database for data conversion to create a PMS dataset for HDM-4. However, due to the low reliability of registered data and system problems, the database system was no longer operable (SAPI2, 2009) [2].

5.2.5.3 Lessons Learned from the Past Application of PMS

Road maintenance long-term and middle-term planning systems in Vietnam have not been operable recently because of the complexity of operation, poor customization features, low database reliability, and system troubles with the database software (Thao, 2013) [16]. From these long trials and application in the past, valuable lessons have been obtained which will be described below.

(1) PMS Should be Changed from Commercial and Black-box based to Open-source Based

PMS must be fully understood as one system with a macro scope of application that reflects many national issues in terms of technical, institutional, policy, budget, etc. Hence, PMS development must be dealt with in a very different manner from common commercial software. Thanks to the recent dramatic improvement of informatics technology, development of a new PMS system specifically for road administration has been confirmed to be the best solution.

Regarding technical issues, the PMS system seems to be very sophisticated, in addition to many potential changes leading to very high demand for improvement and customization of the system during utilization in order to support proper management and maintenance of road

asset infrastructure. In this case, the copyright in commercial systems or source code in black-box or closed systems becomes a very serious issue that can make a critical barrier to any desire of system improvement. In light of this situation, an open-source system is the obvious choice.

(2) Improvement of Involvement and Collaboration among Relevant Stakeholders and Public Announcement

In terms of improving organizations in various fields, research and development (R/D) always plays a significant role. However, in Vietnam, it must be understood that the R/D function in many organizations and agencies, especially in domestic businesses, is rather poor, which seems to be the main barrier to improvement. Given such a situation, there is a need to enhance collaboration among relevant organizations, especially among state management agencies, businesses, and research organizations such as universities and institutes. Application and utilization of previous PMS systems are quite limited for some organizations, without sufficient collaboration with R/D agencies to sustainably absorb technologies.

Moreover, it also has proved difficult to find key engineers who are well acquainted with the operation of the previous PMS systems. The combination of insufficient dissemination of recipient organizations and poor involvement on the part of competent agencies has led to a more challenging situation in terms of making the systems the officially adopted ones. HDM-4 had been used as a tool to create road maintenance plans for submission for approval of budget plans. However, there are no strong or persuasive arguments to make financial agencies recognize the method of budget simulation used in HDM-4 because, in general, they had very limited information on, or understanding of it.

(3) Training is Indispensable to Improve Human Capacity

Unlike common computer software, specialized systems like PMS require training in operation, as they are specialized software for road management. With projects for introduction of these systems, training during project implementation has been provided by donors' consultants. Therefore, after the projects, there was poor expansion of training or dissemination of knowledge because of the lack of local trainers as well as training policy.

(4) Formulation of PMS Dataset Should be Improved in both Methodology and Technology

Right at the beginning of HDM-4's introduction to Vietnam, there was much confusion about the formulation dataset. To cover the very broad objectives, the structure of the dataset is also rather complicated and very large. For each road section or sample, 159 data items (SAPI2, 2009) [2] are requested to be filled manually into the dataset. Collection methods for these data items are very different, without guidance on how to acquire them. The classification method

for these items was also quite simple, without any consideration for hierarchical levels of data, which lead to the critical requirement of acquiring all these data items at the same time.

Moreover, without database functions in the system like data verification or data input control, it was very hard to verify the quality or reliability of datasets.

New technologies in data collection had not been introduced or applied, so it was quite time and cost consuming to formulate the requested datasets.

5.3 NEW APPROACH TO INTRODUCE PMS IN VIETNAM

5.3.1 Global Trends in PMS

With the availability of HDM-4, not only Vietnam but also many other countries in the world have tried to apply this system since the early 21st century (Do and Kwon, 2009) [17]. However, after studies and trial applications, they faced problems of unfeasibility in data preparation and management, impossibility of calibration (Do and Kwon, 2009) [17], untouchable system and so forth that were seriously taken into account for decisions.

In 2009, an international seminar on asset management implementation in Asian countries was held in Malaysia with rich involvement from many road authorities, policy makers, practitioners, researchers from 19 countries and donors' representatives to share current practices of asset management systems and discuss relevant issues, as well as to share common direction and perspective in the future. Among many intensive discussion topics, it was strongly emphasized that the schemes and local conditions of asset management totally differ from country to country. Therefore, one-size-fits-all systems are incapable of dealing with demands for customization. Participants from many countries shared agreement for the direction of asset management systems by approaching to new systems expected to be developed with efforts to make them usable, practical and effective. One-finds-one's-own-size systems seem to be the best solution. However, for the Asian region, the systems can be initiated from one common platform.

Among the countries, high and clear motivation and clear vision for development of new PMS systems was found, including some preparations underway in countries such as: Thailand, Korea, and Vietnam. As in Japan, studies on PMS had been executed actively; for instance, Jido, *et al.* (2004) developed a pavement management accounting system for the road administrators of local governments to execute rational repairs by analyzing the road pavement asset management information system [18].

5.3.2 Approaches of PMS Software System

Being aware of the significance of road infrastructure management and the maintenance to make it be successfully transferred from generation to generation, much attention has been paid since early on. Introduction and application of road management systems like HDM-4 started in the 2000s. Many efforts for system operation and application had been made, including financial investment from state budgets and Official Development Assistance (ODA) funds since the last decade, but the outcome is far from expectations due to both subjective and objective causes, especially the low customization of these "closed" systems due to local conditions or requirements. The lack of training and dissemination to improve knowledge and awareness in the field of asset management during introduction of these systems is also one of the main causes that lead to failed application in Vietnam.

Consequently, the implementation of road asset management systems in Vietnam was pushed to such a critical situation that everything was restarted from the beginning after more than one decade. Three approaches had been classified by Uddin (2006) for consideration: (i) keep using commercial ready-made software, (ii) modify existing commercial ready-made software, and (iii) develop a new customized system, as shown in Table 5.11 [19].

The lessons learned prove that with the many advanced technologies available and high demand for customization, the first approach is not a good choice presently. Moreover, it is also less practical to select the second approach because ready-made software is untouchable and users cannot modify it. Hence, the third option seems to be the best choice especially under the situation of many open-source systems being available such as Linux Operating System. However, without experience in development of road asset management systems and also with the complication and sophistication of system features, the possibility for success in the third approach may also be very low.

Table 5.11 Comparison of approaches to PMS system

Evaluation index \ Options		(1) Use ready-made software	(2) Modify ready-made software	(3) Develop new customized system
Application	Cost (budget + time)	⊙ / x	x / Δ	x
	Initial application (effort to develop)	⊙	Δ	x
Operation	Matching current conditions (data)	x	Δ / ⊙	⊙
	Satisfying current objectives (result)	x	Δ / ⊙	⊙
Future issue	Extending future demand (functions)	x	Δ / ⊙	⊙
	Copyright	x	x / Δ	⊙

Notes) "⊙" - Good, "Δ" - Reasonable, "X" – Bad

Figure 5.11 Symposiums and training courses of road infrastructure asset management in Vietnam

Demand for proper road asset management is quite natural, even in developed countries like Japan. However, differences can be saw, and at the beginning, the third option had been decided on in Japan, which has promoted many research studies on it.

The PMS Kyoto model was also applied to study pavement deterioration in Vietnam within the framework of academic collaboration. Results were presented in some international seminars that have been highly appreciated by Vietnamese road administrators, and representatives of the road sector also expressed the desire to develop a new PMS for Vietnam based on the PMS Kyoto model (SAPI2, Thao 2013) [2], [16].

5.3.3 Academic Achievements and Application of PMS in Japan

With remarkable achievements in development of infrastructure stock, great attention has been paid to asset management and maintenance in Japan. R/D functions are strongly encouraged in many central and local organizations, in addition to close collaboration between state management agencies or businesses and universities or research institutes that have promoted many studies on development of asset management systems.

It can be seen that breakthroughs in research on infrastructure asset management started in the early 21st century, and the studies and team are typical of a new approach in development of more systematic deterioration models, the core component for any asset management system. Stochastic deterioration models using Markov theory have been deeply studied for application instead of conventionally deterministic models. Through their research, many sophisticated papers have been published since the 2000s to share and disseminate new approaches.

The Kyoto model is one new pavement management system that has been developed by a research team.

There exist different points of road infrastructure management systems between Japan and Vietnam, especially in terms of involvement of the private sector in road management and

maintenance, and independent demarcation of management schemes among the central government, local governments and private companies. Therefore, PMS can be different from prefecture to prefecture. During application, the Kyoto model has been customized to develop specialized PMSs for many prefectures in Japan.

5.3.4 Implementation of Human Resource Development for Road Asset Management

The initiative to enhance capacity in road infrastructure asset management through training was made in collaboration between Japanese and Vietnamese universities, with ten annual training courses and four international seminars and symposiums (Figure 5.11) of the topic provided continuously for many road practitioners, engineers, and researchers in Vietnam within the collaboration framework (Figure 5.12). Through many discussions, the high demand for development of one new PMS system for Vietnam can be found, with desire for a new approach that should be practical and suitable to local conditions and requirement of customization in Vietnam. With this useful supporting tool, proper decisions can be made with systematic and strategic plans.

It is confirmed that not only engineers and road practitioners but also many researchers and university lecturers in Vietnam have the chance to know and understand new approaches and knowledge in the field of road asset management including introduction of their understanding of the Kyoto model. Much new information has been incorporated into university lectures to disseminate to students that secures for a firm step of human resource preparation.

Research teams also took the effort to do some case studies on pavement and prediction deterioration models in Vietnam. Results from studies using the Kyoto model on road pavement were presented in two symposiums held in May and October 2010, with significant participation from relevant organizations in Vietnam and Japanese organizations. Representatives of the Vietnam road sector appreciated the studies and highly evaluated the feasibility of the system and the possibility to apply it in Vietnam. Based on that, the decision to develop a new PMS system for Vietnam based on the Kyoto model had been made and a proposal has been officially submitted by the Vietnamese government to Japanese government for consideration.

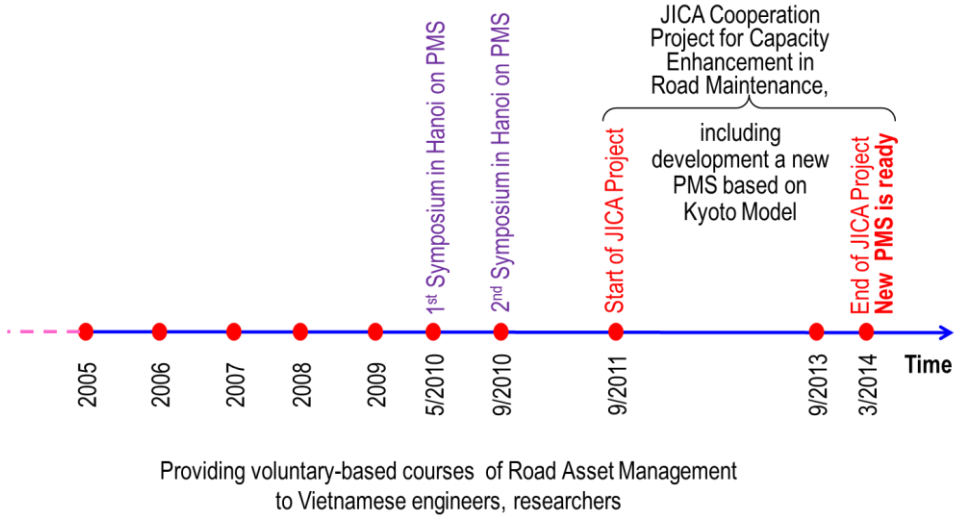


Figure 5.12 Main Activities to Support Enhancing Capacity in Road Maintenance and Management in Vietnam

For the road sector in Vietnam, their approach to the Kyoto model is at the extremely critical threshold of changing direction in PMS systems after the long application of HDM-4. However, it is understood that the more important point for successful implementation of a PMS system in Vietnam is the numerous voluntary-based efforts by Japanese researchers with the aim of enhancing human capacity as the most fundamental preparation for any development.

5.3.5 Formulation of Practical Project in Vietnam for Capacity Enhancement in Road Maintenance

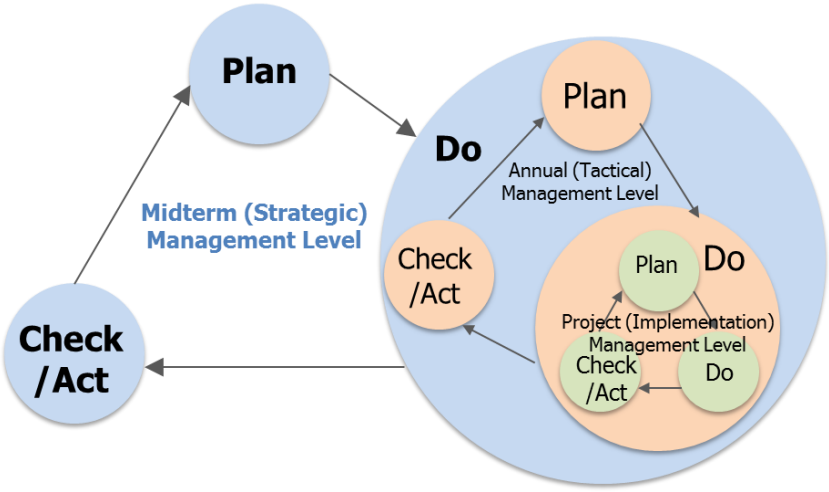


Figure 5.13 Plan-Do-See cycle for Infrastructure Asset Management

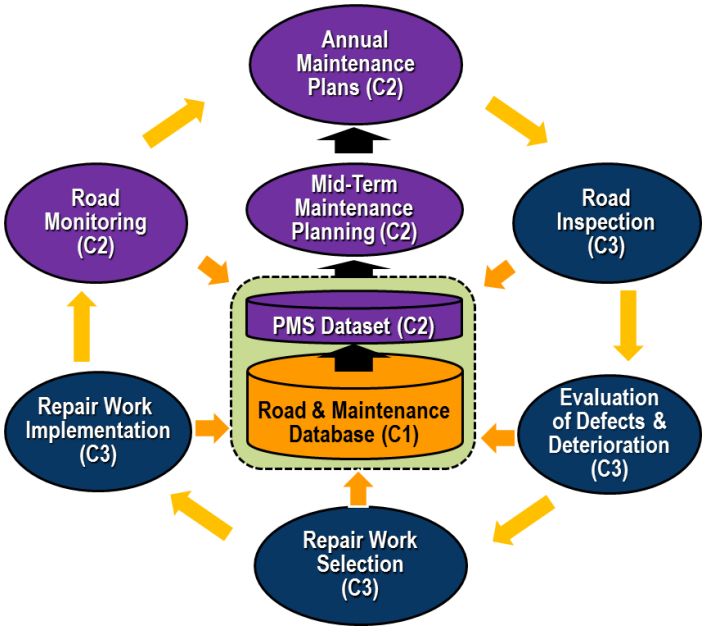
Among many requested proposals from Vietnamese government to Japanese government for consideration to support from fiscal year 2011, proposal to develop a new PMS system for

Vietnam based on the Kyoto model within framework of the cooperation project funded by Japanese government through JICA for capacity enhancement in road maintenance in Vietnam had been selected (MOT, DRVN 2011) with the goal to enhance implementation of PDCA cycle of proper maintenance for road facilities and dissemination project outputs across the country [20].

Definition of PDCA cycle is quite understandable. However, activities breakdown system can be different among road administrators. In the project, hierarchical Plan-Do-See cycle has been selected to breakdown all major activities within the cycle of road asset management and maintenance (Figure 5.13).

All these major activities have been grouped that formulates scope and components of the project. Five components in comprehensive scope of road management and maintenance in the project have been determined (MOT, JICA, 2011) [20]: (i) Enhancement of Road Information Management, (ii) Enhancement of Planning Capacity for Road Maintenance, (iii) Improvement of Road Maintenance Technology, (iv) Strengthening of Road maintenance Institution, and (v) Reinforcement of Human Capacity Development (Figure 5.14). It is quite illustrative to visualize all three first components in the figure. However, the two later components are rather general that is planned to cover the whole cycle.

During the whole project duration from September 2011 to March 2014, dispatched Japanese experts had tightly collaborated with Vietnamese counterpart's members to study on development new PMS system in association with key academic institutions.

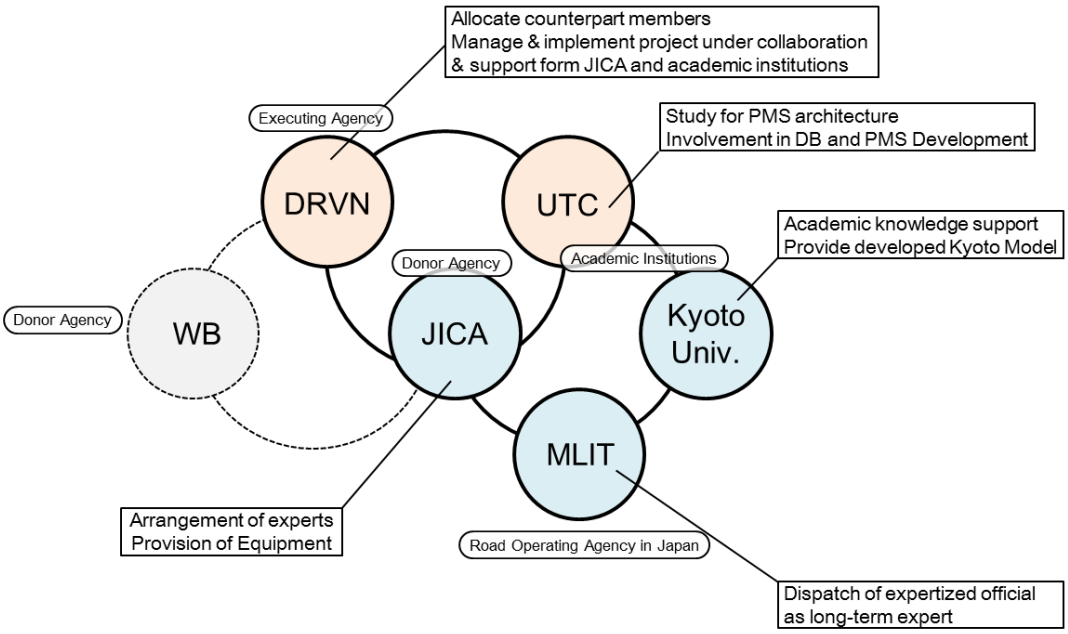


Notes) "C" - project component

Components 4 and 5 have comprehensive scope that covers the whole cycle

Figure 5.14 Road Asset Management Cycle and Scope of Project Components

Among various ODA projects in Vietnam, in this practical project, both sides of Vietnamese road administrator and JICA decided to set up special scheme of cooperation in association with both domestic and oversea academic institutions with the goals to realize the outputs and fully transfer technologies through project implementation process for sustaining project outcomes (Figure 5.15). It had been planned that after the project, Vietnamese engineers should be master in the systems developed in the project not only in operation but in upgrading, expanding or revising the system also.



Notes) DRVN is the counterpart of Project Team in the project including association of both domestic and international institutions (University of Transport & Communications and Kyoto University) and collaboration from the Ministry of Land, Infrastructure, Transport and Tourism of Japan (MLIT). Coordination with other donors is also needed especially for avoiding overlapping.

Figure 5.15 Project Collaboration Scheme

In the scheme, DRVN is the main player since their counterpart role to work with experts in the project team. To challenge the wide and big expectation including development a new PMS system, they received strong support from academic institutions both domestic and oversea especially Kyoto university who has both authorship and ownership of Kyoto model and their rich involvement in Vietnam with UTC on researching and human resource development in the field of infrastructure asset management. Collaboration with other donors and Japanese governmental agencies like MLIT is also one part of the scheme for some relevant issue.

5.4 DEVELOPMENT OF NEW PMS BASED ON CUSTOMIZATION OF KYOTO MODEL WITHIN THE PRACTICAL PROJECT

In the second component of the project, Kyoto model had been selected as the platform for development new PMS system for Vietnam. Based on the past lessons learned, both sides agreed to develop PMS system specialized for utilization under Vietnam conditions based on customization of Kyoto model to support road operators to formulate annual and middle-term pavement maintenance plans on the basis of the pavement condition survey results and statistically analyzed data of future pavement deterioration.

Among various maintenance works, proactive or periodical maintenance and rehabilitation are the targets of pavement maintenance for PMS. Ex-post fact maintenance of routine works and reconstruction should be dealt separately from the system.

Middle-term pavement repair plans are developed for the term varies from three to five years after the latest pavement condition survey. The function simulates the progress of pavement deterioration using Markov hazard model and budget plans over middle-term years and outputs under three maintenance scenarios of maintenance policy under budget constraints.

In parallel with system development, new technology of automatic pavement condition survey has been introduced including provision one survey vehicle to collect pavement performance data such as roughness, rutting, and cracking. Intensive training to transfer the technology to Vietnam engineers has been conducted. The first pavement condition survey had been implemented by trained engineers to formulate datasets with high accuracy and reliability in more productive manner for PMS operation.

5.4.1 Clarification PMS Functions and Formulation of System Modules

5.4.1.1 Road Maintenance Activities on Target

Practices of road maintenance activities are shown in Table 5.12 including consideration of proper activities to be incorporated into the PMS system.

It can be understood that the routine maintenance including non-pavement maintenance and pavement ex-post fact maintenance are targeted for restoring serviceability, but not reducing pavement aging. It is also obvious that the effects of pavement aging reduction are hardly anticipated for ex-post fact maintenance.

In reality, kilometer-based fixed rate of maintenance budget allocation has been widely applied in estimation of budget for annual routine maintenance budgets that can be dealt separately from the pavement maintenance planning system.

In addition, reconstruction in the development works, which comprises widening, realignment and so forth, are carried out mainly for the purpose of enhancement road capacity. Therefore, it also should be handled separately from the pavement maintenance planning system.

As the conclusion, periodical maintenance and rehabilitation are the targeted road maintenance that will be incorporated in pavement maintenance planning system or PMS.

Table 5.12 Road Maintenance Activities on Target

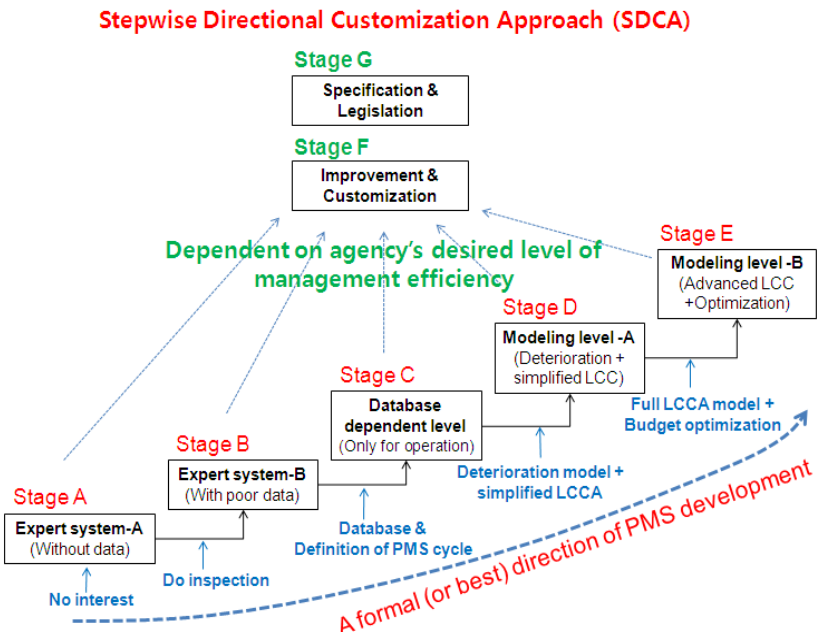
Maintenance Work Categories and Breakdown		**Description of Main Activities** (Based on current Technical Standard of Road Routine maintenance code TCCS:07-2013/TCDBVN)	**Objectives of Measures**			
			Increase Capacity	Increase Strength	Reduce Aging	Restore Serviceability
1	**Routine Maintenance**					
	1.1 Non Pavement maintenance	• Culvert cleaning • Side ditch cleaning • Grading • Grass and bush control				⊙
	1.2 Pavement Ex-post fact maintenance (Reactive Maintenance)	• Pothole patching (shallow) • Pothole patching (deep) • Edge break repair • Gravelling				⊙
2	Unscheduled Maintenance	• Emergency works • Disaster restoration				⊙
3	**Periodic Maintenance** (Proactive Maintenance)	• Crack seal • Single/Double/Triple bituminous surface treatment (SBST, DBST, TBST) • Bituminous penetrated macadam (BPM) • Thin asphalt concrete overlay			⊙	⊙
4	**Development Works**					
	4.1 Rehabilitation	• Fully structural overlay • Pavement replacement		⊙	⊙	⊙
	4.2 Reconstruction	• Realignment • Widening	⊙	⊙	⊙	⊙
5	**New Road Construction**		⊙	⊙	⊙	⊙

5.4.1.2 PMS Functions and Formulation of System Modules

Before the project implementation and out of project framework, various academic efforts on dissemination knowledge of road infrastructure asset management including introduction, interpretation, and operation of Kyoto model within framework of training courses, seminars, symposiums, conferences, etc.,. have been conducted in Vietnam for years. Such kind of in advance implementation made the approach to development new PMS system in the project becomes smoother and in right track.

Implementation of PMS system development seems to be different among countries in the worlds depending on their goals and domestic conditions. To clarify it, Han (2011) had suggested *"Stepwise Directional Customization Approach (SDCA)"* for PMS that is comprised of two phases divided into five general stages and two mature stages as shown in the Figure 5.16 including his further explanation in Table 5.13.

In the SDCA, stage A can be understood as Zero-stage without concept or interest of strategic management and maintenance; all the works are reactive and ex-post fact countermeasures that are totally relied on engineer's judgment and his experience. On the contrary, stage F and stage G are really mature at the top of PMS development process or the ultimate goal of PMS for road administrators.



Source: Han, 2011

Figure 5.16 Suggestion of General Implementation and Improvement Steps of the PMS by the SDCA [21]

Table 5.13 Description of PMS Capability Levels

PMS features	Descriptions	Requirements to advance to next stage	Main functions of PMS (Capability of PMS)	Additional system components	Core data
Stage A	Expert system dependent level without data	Inspections for pavement condition and inventory data	No function and system	N/A	N/A
Stage B	Expert system dependent level with (incomplete) data	• Database + definition of PMS cycle and activities • Securing general data requirement in PMS	(For a PMS cycle) • Maintenance schedule • Inspection schedule • Budget estimation	Data tables	• Pavement condition indices • Minimized inventory data
Stage C	Database dependent level or PMS cycle-oriented level	• Pavement deterioration forecasting model(s) • LCC model for agency cost	(For a PMS cycle) • Overall PMS activities plan • Basic database function • Support external models • Data error processing • Maintenance design	• Database • Internal PMS model • External PMS model (if necessary)	• Additional data to be general dataset • Unit costs by maintenance types

Han				• Budget estimation		

PMS features	Descriptions	Requirements to advance to next stage	Main functions of PMS (Capability of PMS)	Additional system components	Core data
Stage D	Modeling level A - Pavement deterioration forecasting model (Performance analysis + estimation of agency cost)	• Full LCCA models • Optimization functions	(During analysis period) • Performance analysis • Comparison of maintenance strategies • Economic analysis on road agency cost • Accounting function	• Forecasting model (s) • Functions estimating road agency cost	• Enough time-series performance data • Explanatory PMS variables
Stage E	Modeling level B - LCC (road user and socio-environmental cost +optimization)	• Domestication • Customization • Elaboration	• Economic analysis with additional LCC contents • Optimization functions which maximize NPV or condition recovery	• LCC models • Optimization functions	• LCC related data • Unit costs for additional LCC contents • Model coefficients
Stage F	Feedback level	Continues feedback	Feedback, improvement, and customization of current PMS	Case by case	Case by case
Stage G	Specification, documentation and legislation level	Ultimate stage but needs continuous feedback corresponding with new demands	Documentation & application to real field work	Case by case	Case by case

(2011) also studied and classified PMS functions in hierarchical arrangement with brief description for each function. The matrix in Table 5.14 supports road administrators to know how to initiate their first try of PMS development or to identify the location or stage of their current PMS to plan for PMS improvement or upgrading in the SDCA approach.

Table 5.14 Definition of General Functions in PMS [21]

PMS functions	Description of PMS functions in term of sophisticated level				Available in stage(s)
	Core level	Semi-core level	Recommended level	Advanced level	
Data requirements	(For stage A and B) • Identification • Simplified inventory & condition data • Unit costs for agency cost	(For stage C) General inventory that includes pavement condition indices and PMS variables	(For stage D) • Enough time-series pavement condition • (special) PMS variables	(For stage E) • Detail data for LCC modeling • Unit costs for socio-environmental cost • Subsidiary data	A, B, C, D, and E
Database	• Basic database function • Data exporting	Reporting function (network condition, simple statistics)	Support for internal and external models (*e.g.* HDM-4)	• Visualized database • Web-based system • Link with other road facilities or systems	C, D, and E
PMS cycle management	• Finding work demands (inspection & maintenance) • Data error processing	• Work effect models • Work design models • Budget estimation	Reporting function (summary of activities during a cycle)	Near-optimization of agency cost (by changing current plan)	
Pavement deterioration forecasting	Deterministic - I Empirical-mechanistic model (for beginning stage only)	Deterministic - II (*e.g.* Single and multiple regression)	Stochastic - I (*e.g.* Markov hazard model)	Stochastic – II (*e.g.* Local mixture hazard, Hidden Markov chain, etc.)	D, and E
Economic analysis	Minimized LCCA - I Maintenance cost only (by deterministic or stochastic approach)	Minimized LCCA-II User-specified agency cost only (*e.g.* adding inspection cost)	Simplified LCCA Agency cost + (user-specified) simplified road user cost	Advanced LCCA Full road user costs and socio-environmental cost Accounting function	E, partially D
Optimization	Optimization – I Work scheduling by	Optimization-II Maximizing	Optimization-III Maximizing NPV	Optimization-IV Best maintenance strategy	E, partially D

	user-specified priority ranking	condition recovery		by long-term accounting concept	

After long period of first approach to ready-made PMS systems, Vietnamese road administrators strongly be aware of the significance of sustainable approach for PMS development. Since the first approach of Kyoto Model in Vietnam in 2005, Stage C had been selected as the initial step for activities within academic collaboration. So far, Stage E has been applied in researching programs but in practical reality, the common understanding of road practitioners is around Stage C as the results of great effort of dissemination across the country by academic collaboration scheme.

Therefore, for development new PMS system in the project, State C has been selected as shown in double line-border and shaded cells in Table 5.13 and Table 5.14. Of course, some functions can be integrated with advanced models or technologies such as the application of local mixture hazard model for pavement deterioration forecasting function.

Regarding to the common requirements on PMS system on outputs that consists of :

- Results of pavement deterioration prediction or performance curves
- Middle-term pavement maintenance plan and budget simulation including list of repair works
- Annual pavement maintenance plan including list of repair works
- Results of factor analysis and Benchmarking evaluation to find out key factors on pavement deterioration as well as to point out road sections with abnormal deterioration which may need further investigation to find out the causes and solutions.

the software architecture had been designed comprising of one master database and four modules: (i) Data Management Module, (ii) Pavement Deterioration Evaluation Module, (iii) Repair Work Planning Module, and (iv) Budget Planning Module (Figure 5.17).

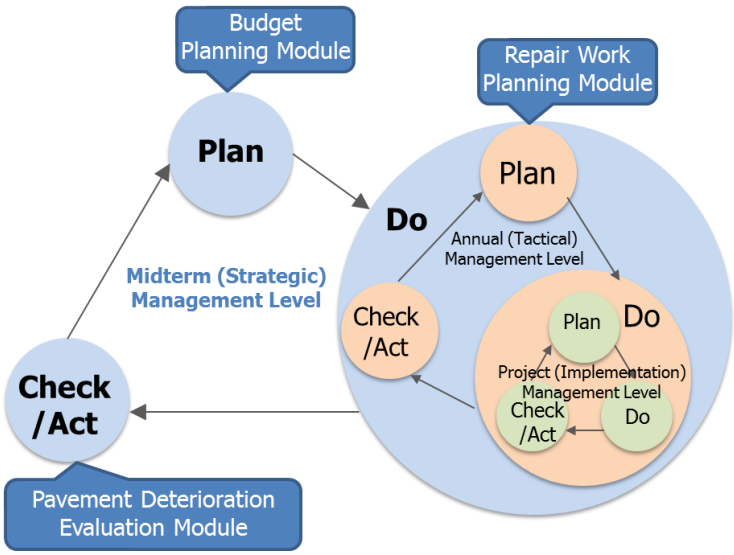


Figure 5.17 Illustration of System Modules (excluding Data Modules) in PMS System

Master database plays a core role in recording database information as well as conditional information needed for the computation in other modules, including those inputted by operator or those are automatically generated through the computation processes.

Figure 5.18 shows general system flow of PMS system.

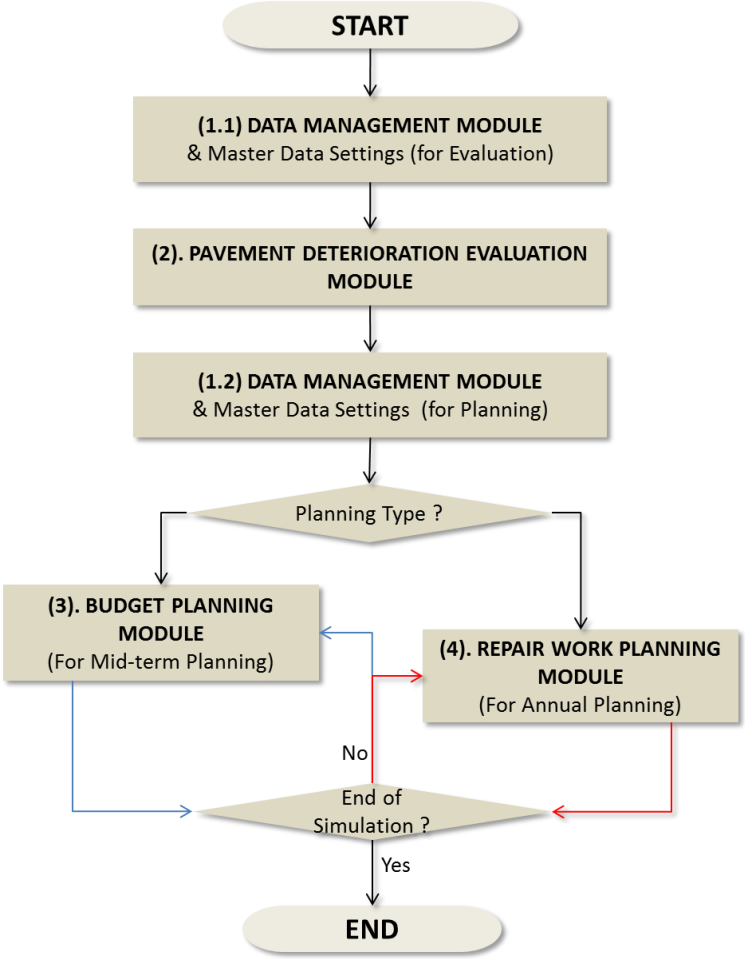


Figure 5.18 General system flow of PMS system

Data management module imports and updates PMS dataset into the system, select proper data items from the imported PMS dataset and internally generates data to formulate individual datasets for three other modules (Figure 5.19).

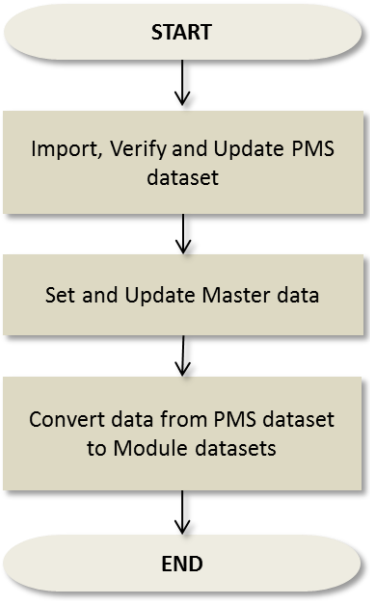


Figure 5.19 General system flow and functions of Data Management Module

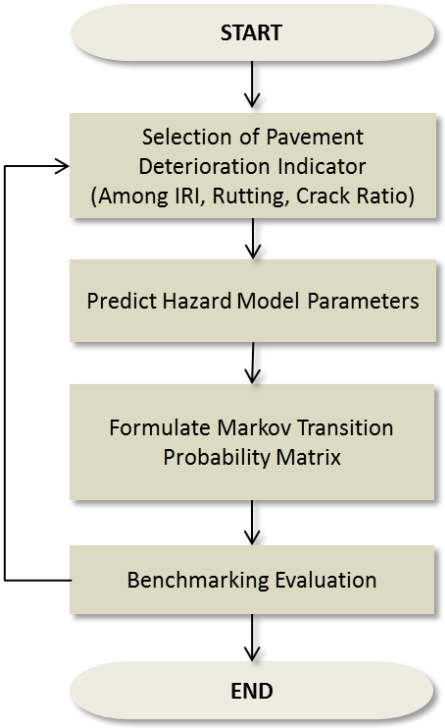


Figure 5.20 General system flow and functions of Pavement Deterioration Evaluation Module

Pavement deterioration evaluation module analyzes pavement deterioration progress or the transition of pavement condition states or its pre-defined rankings. Markov transition probability theory is applied in the analysis to calculate the transition probabilities from a certain ranking to other rankings of pavement deterioration based on time-series data of periodically surveyed pavement conditions to specify pavement deterioration rate and its life expectancy in each ranking and in whole.

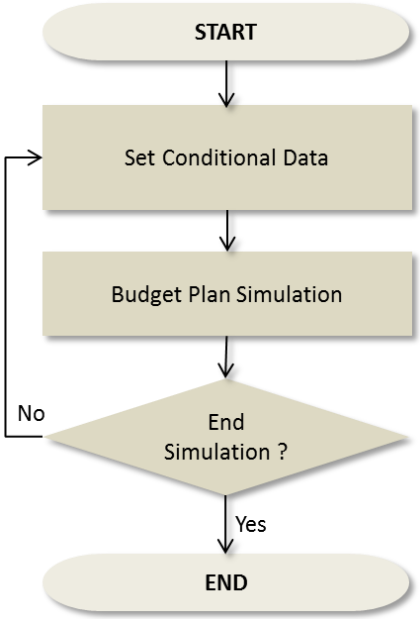


Figure 5.21 General System Flow and Functions of Budget Planning Module

Budget planning module simulates maintenance scenarios to formulate middle-term pavement repair plans that should be submitted to competent decision makers for approving with the focus point of budget preparation or registration. In practices of pavement management, duration of three years is proper for middle-term.

While budget planning module focuses on strategic maintenance, repair work planning module formulates pavement repair plans for a single year incorporating the latest pavement condition survey data. Based on repair standard in general or flowchart of work repair selection in particular, repair candidate sections are clarified in right priority including recommendations of repair methods that must be used to complete cost estimation of annual repair plans.

Figure 5.22 shows general system flow and functions of repair work planning module.

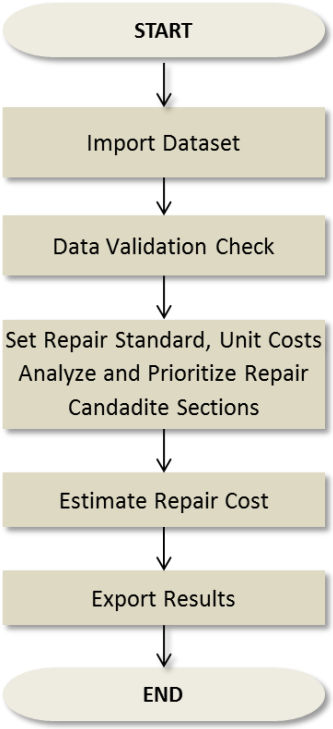


Figure 5.22 General System Flow and Functions of Repair Work Planning Module

Detail of computation flows for all these system modules are described in detail in Section 5.4.4.

5.4.2 Flowchart of Pavement Work Repair Selection

5.4.2.1 Selection of Pavement Indicators and Pavement Condition Indices

Regarding to pavement performance, it can be characterized by various indicators such as: roughness and rutting that show pavement deformation resistance, skid resistance for surface texture, cracking that has strong relation with structural failure in fatigue, etc. In more detail, for pavement structural evaluation especially at project or implementation level, information of pavement loading capacity or strengths of materials are also necessary.

Determination pavement indicators depends on objectives of information utilization as well as availability of proper technologies, and available resources for data collection specially in case of huge data for all road network.

Taking into account of current practices of pavement management and maintenance in Vietnam in combination with studying other oversea practices, three pavement indicators of roughness in IRI, rutting depth, and cracking ratio have been selected to incorporate in the new PMS system.

Moreover, just using pavement individual indicator for overall evaluation its quality or soundness is insufficient because such individual value cannot express the comprehensive health of pavements. Without comprehensive health index, the possibility for evaluating, in a unified fashion, pavement condition for different routes, regions, among various road administrators, and so forth in the whole road network is quite low. Therefore, it is necessary to formulate pavement index calculated or converted from pavement distress information. Combination of information from pavement distresses, its severities, expansion or quantities into one single number is the common approach to formulate pavement composite condition index that can be used for many purposes such as: (i) evaluation road network, (ii) determination the needs for maintenance in priority, (iii) pavement deterioration evaluation.

In reality, such pavement indices are quite popular in many countries. In this section, typical indices of PCI (Pavement Condition Index), PSR (Present Serviceability Rating) in United States of America (USA) and MCI (Maintenance Control Index) in Japan are discussed as follows.

(1) Present Serviceability Index - PSI

Among three mentioned pavement indices, the PSR was developed at first based on the findings of the AASHO Road Test in USA occurred from 1956 to 1961. At such road test, the concept of pavement serviceability evolved from the concept that the prime function of a pavement was to serve the traveling public (HRB, 1962) [22]. The concept of pavement serviceability developed at the road test is a measure of the ability at the observation time of a certain pavement to serve the traffic that uses the facility (John P. Hallin *et al*, 2007) [23].

At the road test, pavement rating had been conducted by a panel of raters consisting of both automobile and truck drivers for 138 pavement sections in three different states in USA (Carey and Irick, 1962) [24]. Pavement rating scale was defined that consists of five rankings to guide the panel to rate each section during travelling on the road (Table 5.15). These scores or numerical ratings had been referred to as pavement present serviceability rating.

Table 5.15 Pavement rating scale in term of PSR of the AASHO Road Test

No.	Rating scale	Score
1	Very poor	[0; 1)
2	Poor	[1; 2)
3	Fair	[2; 3)
4	Good	[3; 4)
5	Very good	[4; 5]

It can be understood that the panel rating is quite subjective manner. Therefore, in parallel with the panel rating, professional road test crews were also mobilized to execute physical measurements of pavement performance or condition at the same time of panel rating implementation (John P. Hallin, 2007) [23]. Pavement longitudinal roughness, cracking, patching and rutting are the targeted measurements for flexible pavements. Once completion both panel rating and physical measurement, regression analysis were applied with the obtained data to find out relationship between numerical ratings of PSR and measured indicators of pavement conditions for development present serviceability index (PSI) that can be calculated based on pavement indicators as the following equation originally developed from the ASSHO Road Test.

$$PSI = 5.03 - 1.9 \times log(1 + SV) - 1.38(RD)^2 - 0.01 \times \sqrt{(C + P)} \quad (5.1)$$

Where,

- PSI: Pavement present serviceability index
- SV: Slope variance over section
- RD: Pavement rut depth (inch)
- C: Pavement cracking ratio (ft / 1000 ft^2)
- P: Pavement patching ratio (ft^2 / 1000 ft^2)

When IRI test become available, conversion formulas had been proposed to calculate SV from IRI for convenient computation.

(2) Pavement Condition Index - PCI

The PCI for roads and parking lots was originally developed by the U.S. Army Corps of Engineers. During fiscal years 1975-76, at the Air Force's request, the U.S. Army's Construction Engineering Research Laboratory instituted one study on objective and accurately method to evaluate airfield pavement (Shahin *et al*, 1977) [25]. As the result, pavement condition index rating system had been developed based upon ultimate outcomes of such research. It had been standardized by ASTM to supplement to ASTM standard systems for common utilization in USA. Current edition was approved on Dec. 1, 2007 under the fixed designation D 6433 in combination with the number following immediately the designation to indicates the year of original adoption or the year of last revision (ASTM, D 6433-07) [26].

The PCI is a numerical indicator that is used to rate pavement surface condition based upon its exposed distresses or defects with the range of score from 100 for the best condition to zero for the worst performance. The PCI also implies the integrity of pavement structure but not its structural loading capacity.

According to D 6433-07, nineteen types of asphalt concrete pavement distresses (alligator cracking, bleeding, block cracking, bumps and sags, corrugation, depression, edge cracking,

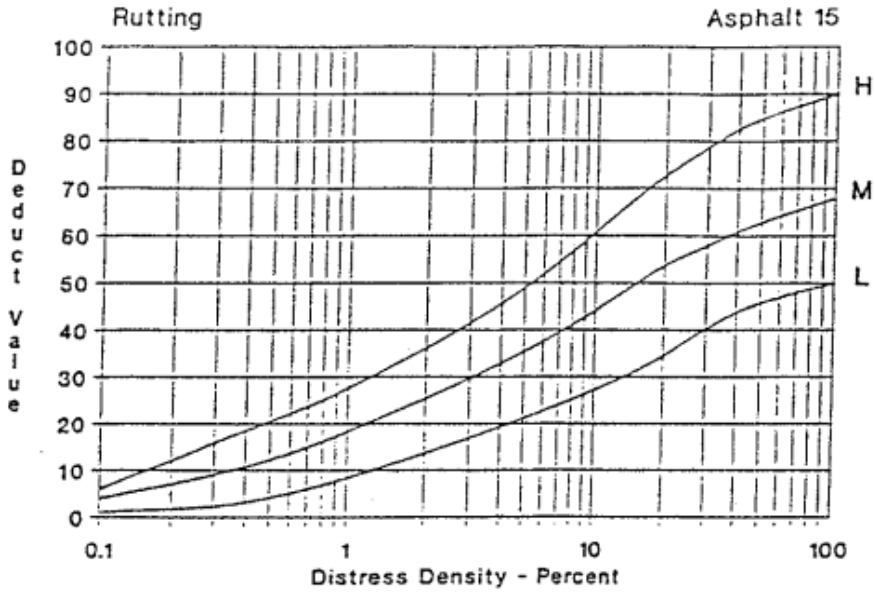
reflection cracking, shoulder drop off, line cracking, patching, polished aggregate, pothole, railroad crossing, rutting, shoving, slippage cracking, swell, and raveling) with its severity are defined that can be used for pavement condition assessment by visual inspection. Each type of pavement distress or defect takes a certain reduction to pavement soundness that is denoted by deduct value (DV). Once visual inspection data is available, individual DV for each distress can be determined by simple projection and interpolation using ready-made corresponding nomogram (Figure 5.23). Similar approach is also applied for cement concrete pavement to rate its conditions.

The PCI is determined by subtracting the maximum corrected deduct value (CDV) that is calculated from all individual DVs from pavement maximum value of PCI.

$$PCI = PCI_{max} - CDV_{max} \tag{5.2}$$

Regarding to the definition of PCI and its range of value, the above formula can be re-wrote as follow

$$PCI = 100 - CDV_{max} \tag{5.3}$$



Source: ASTM standard, D 6433-07

Figure 5.23 Nomogram for Determination of Deduct Value of PCI in term of Rutting Distress

The calculated PCI gives objective and rational information of pavement conditions that can be the basis for specifying the needs of pavement maintenance and repair. More over for strategic maintenance, it is also possible to evaluate pavement deterioration progress by analyzing time-series PCI data.

(3) Maintenance Control Index - MCI

In 1981, Japanese Ministry of Construction firstly proposed and introduced the method to calculate MCI value as the results of three-consecutive-years research program on "Planning of Pavement Maintenance and Rehabilitation". There are four formulas for MCI calculation as shown in following equations [27]. The smallest value of MCI is applied as representative MCI value for each pavement section.

$$MCI = 10 - 1.48 \times C^{0.3} - 0.29 \times D^{0.7} - 0.47 \times F^{0.2} \quad (5.4)$$

$$MCI_0 = 10 - 1.51 \times C^{0.3} - 0.3 \times D^{0.7} \quad (5.5)$$

$$MCI_1 = 10 - 2.23 \times C^{0.3} \quad (5.6)$$

$$MCI_2 = 10 - 0.54 \times D^{0.7} \quad (5.7)$$

Where,

- *C=Pavement cracking ratio in percentage*
- *R=Pavement rutting depth in mm*
- *F=Jyuudan Outotsu Ryou (Traverse unevenness volume or "Heitan sei" in Japanese) in mm.*

Practically, MCI has been widely used for evaluation and identify the needs, priority of pavement rehabilitation by Japan's road administrators at both central level and local level.

(4) Selection of Pavement Index in Vietnam

Physical measurement or surveys of pavement conditions in Vietnam have been conducted many times for both road network level and project level. There are many collected data of pavement conditions including structural bearing capacity especially at project level but pavement conditions are indicated by only individual indicators that leads to some difficulties in unified evaluation the pavement soundness.

Development one pavement index for road sector in Vietnam must be one of prioritized tasks and there are some options can be considered. Try to follow the way to develop known indices such as PSI, PCI, or MCI is one option. However, in any case of formulation these indices is very empirical and practical that reminds for another practical approach to select one of the available indices in reality as the first step. Customization the selected index during implementation of road maintenance and management to make sure that it becomes suitable to the local condition should be the next effort.

For simple comparison some available pavement indices, seven criteria of comparison have been proposed as shown in Table 5.16.

Table 5.16 Comparison some available Pavement Indices

No.	Items	PSI	PCI	MCI
1	Meaning	More riding comfort-based	Condition-based	Condition-based
2	Range of Value	[0 - 5] Narrow range	[0 - 100] Wide range	[0 - 10] Normal range
3	Calculation Formula	Available	Available But need Ready-made Nomograms for all distresses	Available
4	Parameters	3 indicators (IRI, Rutting depth, Cracking ratio)	19 parameters (for AC) 19 parameters (for CC)	3 indicators (IRI, Rutting depth, Cracking ratio)
5	Data demand	Normal	High demand	Normal
6	Possibility of Data Automatic Collection	High	Low	High
7	Uses			
-	Rating pavement condition	Yes	Yes	Yes
-	Specify necessary of repair work	Yes	Yes	Yes
-	Make flowchart of Repair work selection	Not so practical	Yes	Yes
-	Possibility of Customization	Yes	Hard (depend on Ready-made Nomograms)	Yes

Taking consideration all information listed in the table, MCI has been proposed for selection as the pavement condition index in Vietnam due to its simplicity in both data preparation and processing, and utilization. Pavement index is just a numerical indicator but it also reflects more or less the political viewpoint of decision makers on road maintenance policy. At the current situation of national social-economic, such condition-based index can be applied to realize maintenance policy with the emphasis on securing basic functions of public infrastructure in general or road pavement in particular. In the next developed stage, expansion of function requirement seems to be more suitable and road users' convenience-based approach should be considered to upgrade or modify pavement index.

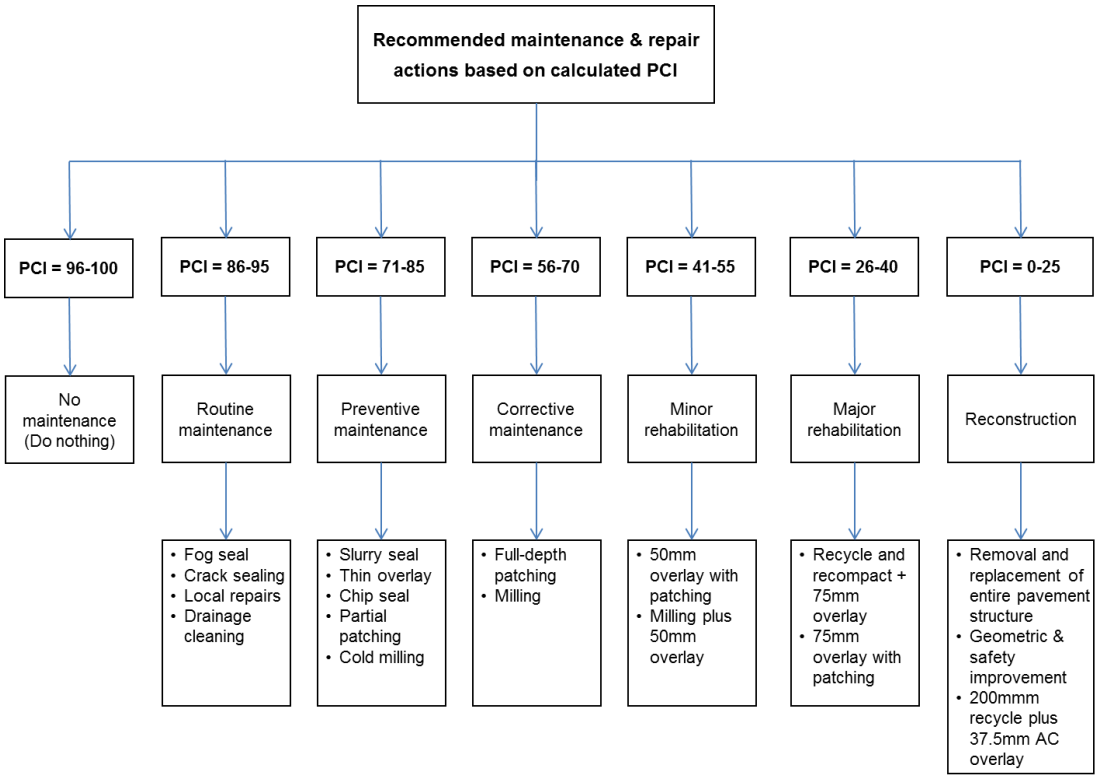
5.4.2.2 Flowchart of Pavement Work Repair Selection

(1) Some Practices of Pavement Work Repair Selection

There are various applications of pavement condition indicators and its index in road management and maintenance. In the section, two cases of practices of pavement work repair selection are discussed for the direction to develop a flowchart of pavement work repair

selection that is planned to incorporate into new PMS system in Vietnam. In 2009, A.S. Adedimila *et al*., suggested one flowchart shown in Figure 5.24 for Nigeria application.

PCI values are grouped into seven classes as shown in the figure. Pavement sections with PCI value exceeding 86 have very good surface condition that only require for routine maintenance. Pavement with PCI in the range from 56 to 85 is found to have a rather good surface condition requiring only corrective maintenance or minor repair. To prolong pavement life, it is also recommended to execute preventive maintenance for this state. Pavement sections with PCI values in the range from 41 to 55 are found to have fair surface condition and minor rehabilitation or medium repair is requested. The poor condition of pavement with the range of PCI from 26 to 40 should require for major rehabilitation or big repair. In the worst case, reconstruction is indispensable for pavement with PCI less than 26. In each maintenance, typical measures are also recommended [28].



Source: A.S. Adedimila, 2009

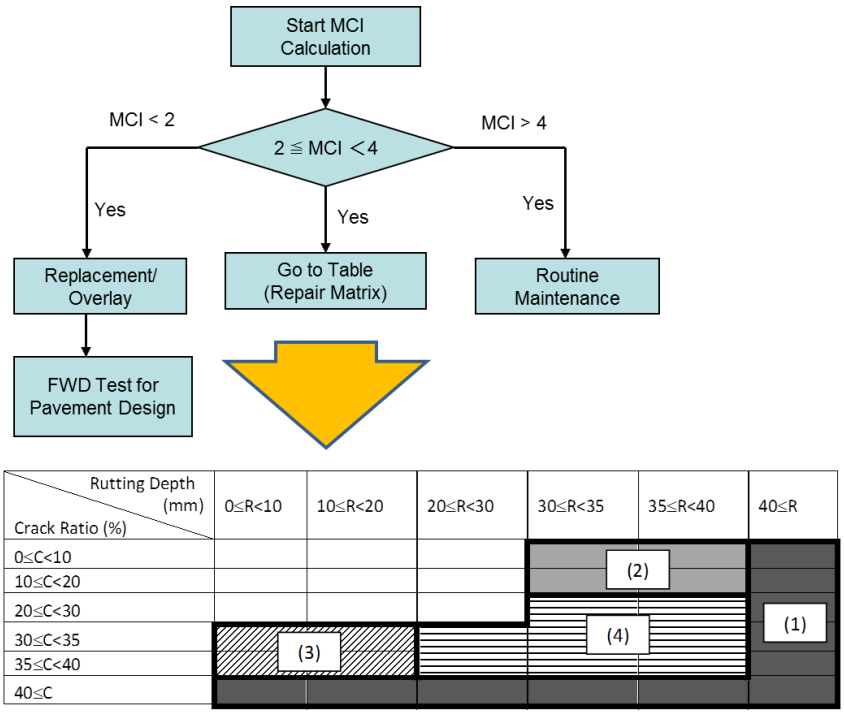
Figure 5.24 Suggested Maintenance & Repair Feasible Actions for Flexible Pavements in Nigeria

In case of Japanese practices in road maintenance and management, flowchart of pavement work repair selection had been developed in MLIT for finding out proper pavement treatment corresponding to its conditions that is characterized by both overall index of MCI and individual indicators of crack ratio and rutting depth (Kanoshima, 2013) [29].

Based on calculated MCI, pavement maintenance is divided into three groups: (i) routine maintenance is applied for pavement in good condition with MCI value exceeding 4, (ii) repair works are recommended for pavement in fair condition with MCI value in the range from 2 to 4, (iii) replacement or reconstruction should be the unique countermeasure for pavement in bad condition with MCI value less than 2 (Figure 5.25).

There are four categories of repair works in the second group that are specified by referring to the repair selection matrix taking into account of two typical distresses in terms of pavement crack ratio and rutting depth.

In case of deteriorated pavement that requires for replacement or reconstruction, further structural investigation need being conducted to specify the proper structure for each treatment.



Notes: groups of repair work recommendations

(1) Repair: Cut and Overlay

(2) Proactive Repair: Cut Work

(3) Proactive Repair: Crack Sealing

(4) Proactive Repair: Crack Sealing &Cut Work

Figure 5.25 Flowchart of Pavement Work Repair Selection - Practices in MLIT

(2) Development Flowchart of Pavement Work Repair Selection in Vietnam

In Vietnam, pavement repair selection has been decided by engineers based on their own judgment and experience. Annually, to prepare for road maintenance plans in the coming fiscal year, field forces and maintenance contractors are requested to visually check pavement

conditions for roads under their jurisdiction to make the list of candidate repair sections. Based on the submitted lists, road administrators in headquarters arranges their mission to verify proposals in the field for finalization the proposals of candidate repair sections including conclusion of repair works. Figure 5.26 shows one example of typical repair works for asphalt concrete pavement and subjective repair selection. More details of the practices are shown in Table 5.17 as typical example of common pavement repair works. There has been no criteria for repair selection. In case of rehabilitation and upgrading, loading capacity of existing pavement is requested to investigate for designing new pavement structure.

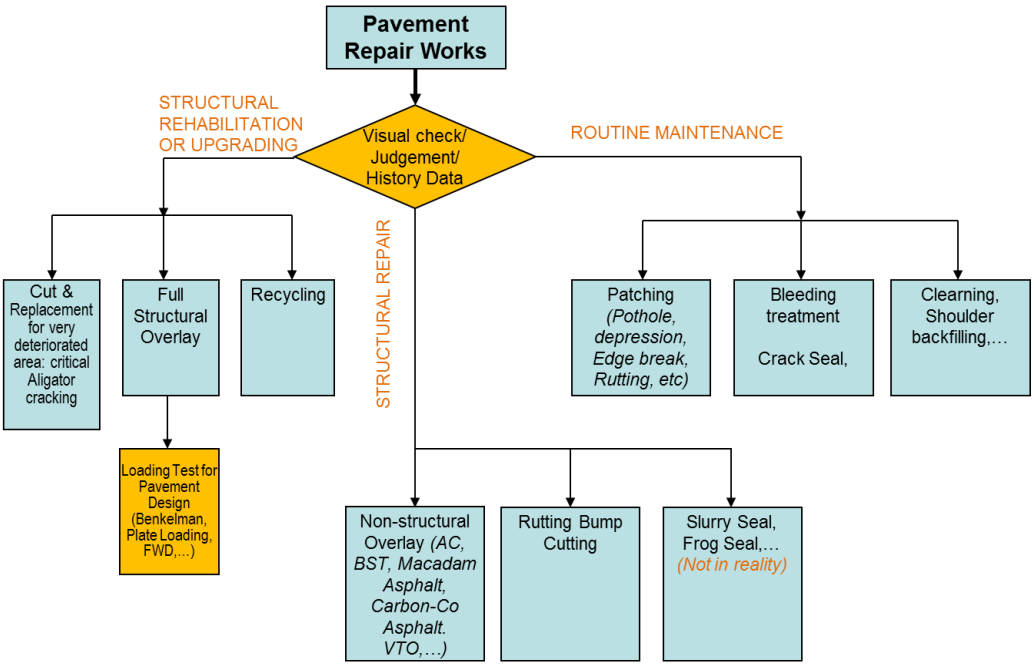


Figure 5.26 Current Practice of Subjective Work Repair Selection for Asphalt Concrete Pavement in Vietnam

Due to a variety of causes in serious condition especially the violation of overloaded vehicles, pavement distresses are also complicated that leads to the diversified countermeasures of treatment. One questionnaire survey was conducted to find out the current practices of pavement maintenance and repair. Questionnaire sheets were distributed to road administrators and maintenance practitioners. Regarding to four main types of pavement defects: (i) pavement deformation, (ii) pavement rutting, (iii) subgrade deterioration and plastic deformation, (iv) pavement structural deterioration, Table 5.17 shows common repair treatment in reality.

Table 5.17 Common Pavement Repair Works

No.	Pavement Defects	Repair Method		Remarks
1.	**Pavement Deformation (including Cracks) with depth in average:**	**Pavement Type**	**In detail**	
		Filling works for Isolated/pot treatment		
	D <= 2.0cm	DBST	DBST	
	D <= 2.5cm	DBST	DBST	
	D = 4cm	DBST	DBST	
			Macadam thin layer for leveling (1.5cm thick)	
			Tack coat (0,8kg/m2)	
	D = 5cm	DBST	DBST	
			Macadam thin layer for leveling (2.5cm thick)	
			Tack coat (0,8kg/m2)	
	D = 6cm	DBST	DBST	
			Macadam thin layer for leveling (3.5cm thick)	
			Tack coat (0,8kg/m2)	
	D = 7cm	TBST	TBST	
			Macadam thin layer for leveling (3.5cm thick)	
			Tack coat (0,8kg/m2)	
	D = 8cm	TBST	TBST	
			Macadam thin layer for leveling (4.5cm thick)	
			Cleaning existing surface & Scratching	
	NA	AC	Fine (Dmax=12,5mm) Dense AC (5cm)	
			Tack coat (0,8kg/m2)	
			Leveling by Fine Dense AC (ex, ~1.5cm)	
		Cutting Asphalt Layer & Replacement		
	D = 5cm	AC Replacement	Fine (Dmax=12,5mm) Dense AC (5cm)	
			Prime coat (0,8kg/m2)	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
	D = 7cm	AC Replacement	Fine (Dmax=12,5mm) Dense AC (7cm)	
			Prime coat (0,8kg/m2)	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
		Thin Overlaying		
	NA	AC thin overlay	AC (5cm)	No calculation for overlay structure, just apply thin layer of AC to improve surface condition based on judgment & visual check at site & confirmation by video at office taking consideration of pavement repair history.
			Tack coat	
			Leveling (by AC for thin leveling, by SBST or DBST for thick leveling)	
	NA	AC thin overlay	AC (7cm)	
			Tack coat	
			Leveling (by AC for thin leveling, by SBST or DBST for thick leveling)	

No.	Pavement Defects	Repair Method		Remarks
	NA	TBST/ DBST	TBST/DBST	Hot bitumen is very popular. Recently, it has been encouraged to use emulsion instead of hot bitumen for some sites
			Macadam thin layer for leveling	
			Tack coat	
2.	**Pavement Rutting with depth:**			
		Filling works (in combination with Cutting humps due to rutting)		
	D = 6cm	DBST	DBST	
			Macadam thin layer for leveling (3.5cm thick)	
			Tack coat (0,8kg/m2)	
	D = 8cm	TBST	TBST	
			Macadam thin layer for leveling (4.5cm thick)	
			Cleaning existing surface & Scratching	
	NA	BPM	BPM	BPM has not been popular in comparison with BST because it is impossible to control the penetrated depth of bitumen in macadam layer. It is also not so cheap as BST.
			Macadam thin layer for leveling	
			Cleaning existing surface & Scratching	
		Structural Cutting & Replacement		
	NA	AC - Surface	Fine (Dmax=12,5mm) Dense AC (5cm)	
			Tack coat (0,5kg/m2)	
			Middle (Dmax=19mm) AC (5cm)	
			Prime coat (1kg/m2)	
			Compacting existing base	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
3.	**Totally Deteriorated & Plastic Deformation in Subgrade**	**Pavement Replacement**		
		AC - Full structure	Fine (Dmax=12,5mm) Dense AC (5cm)	For isolated (spot) treatment: no calculation for replacement structure, just try to apply similar structure with original one.
			Tack coat (0,5kg/m2)	
			Middle (Dmax=19mm) AC (5cm)	
			Prime coat (1kg/m2)	
			Base course (Aggregate)	
			Sub-base course (Aggregate)	
			Subgrade (~30cm)	
			Compacting existing ground	
			Excavating Existing Subgrade	
			Excavating Existing Foundation (Base & Sub-base)	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
		TBST - Full structure	TBST	For isolated (spot) treatment: no calculation for replacement structure, just try
			Macadam Base	
			Subgrade (~30cm)	

No.	Pavement Defects	Repair Method		Remarks
			Compacting existing ground	to apply similar structure with original one.
			Excavating Existing Subgrade	
			Excavating Existing Foundation (Base & Sub-base)	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
4.	**Structural Deteriorated Pavement**			
			Pavement Replacement	
	Serious condition but in isolated (spot) locations	BST Replacement in spots	TBST	No calculation for replacement structure, just try to apply similar structure with original one. Replacement by BPM is not popular.
			Macadam layer	
			Compacting existing base	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
		AC Replacement in spots	AC surface (1 or 2 layers)	
			Prime coat (1kg/m2)	
			Foundation (Base or including Sub-base)	
			Compacting existing base	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
	Serious condition but in wide scope	BST Replacement in wide scope	TBST	Need for loading capacity testing to determine existing pavement strength. Based on requested strength (identified at original design phase), replacement structure is analyzed and calculated to recover structure capacity. Replacement by BPM is not popular.
			Macadam layer	
			Compacting existing base	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
		AC Replacement in wide scope	AC surface (1 or 2 layers)	
			Prime coat (1kg/m2)	
			Foundation (Base or including Sub-base)	
			Compacting existing base	
			Excavating Existing Asphalt Layers	
			Cutting Existing Asphalt Pavement	
			Thick Overlaying	
	Serious condition but in isolated (spot) locations	AC Replacement in spots	AC surface (1 or 2 layers)	No calculation for replacement structure, just try to apply similar structure with original one.
			Prime coat (1kg/m2)	
			Foundation (Base or including Sub-base)	
	Serious condition & in wide scope	AC Replacement in wide scope	AC surface (1 or 2 layers)	Need for loading capacity testing to determine existing pavement strength. Based on requested strength (identified at original design phase), overlay structure is analyzed and calculated to recover structure capacity.
			Prime coat (1kg/m2)	
			Foundation (Base or including Sub-base)	
			Recycling	
	Serious condition & in wide scope	AC Replacement in wide scope	New AC surface (usually 1 layer)	Need for loading capacity testing to determine existing pavement strength. Based on requested strength (identified at original design phase), new recycled structure is analyzed and calculated to recover structure capacity.
			Recycling asphalt surface	

With less constraint on pavement elevation and resource of natural materials, overlaying have been applied widely for both medium and big repair works. Pavement cutting and replacement, and recycling are quite rare, only for critically deteriorated parts. Common repair works for pavement in Vietnam can be classified into seven categories as shown in Figure 5.27 and Figure 5.28.

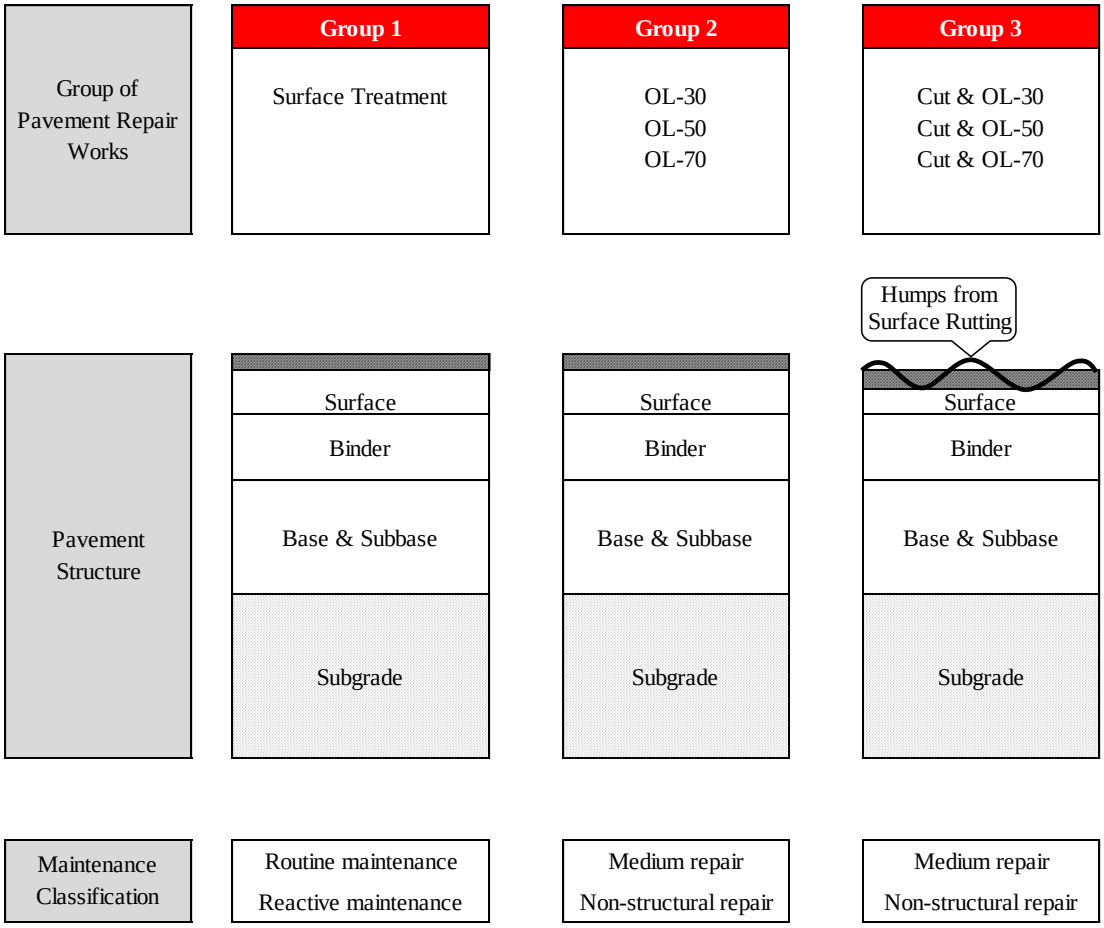


Figure 5.27 Common Repair Works for Pavement in Vietnam (1/2)

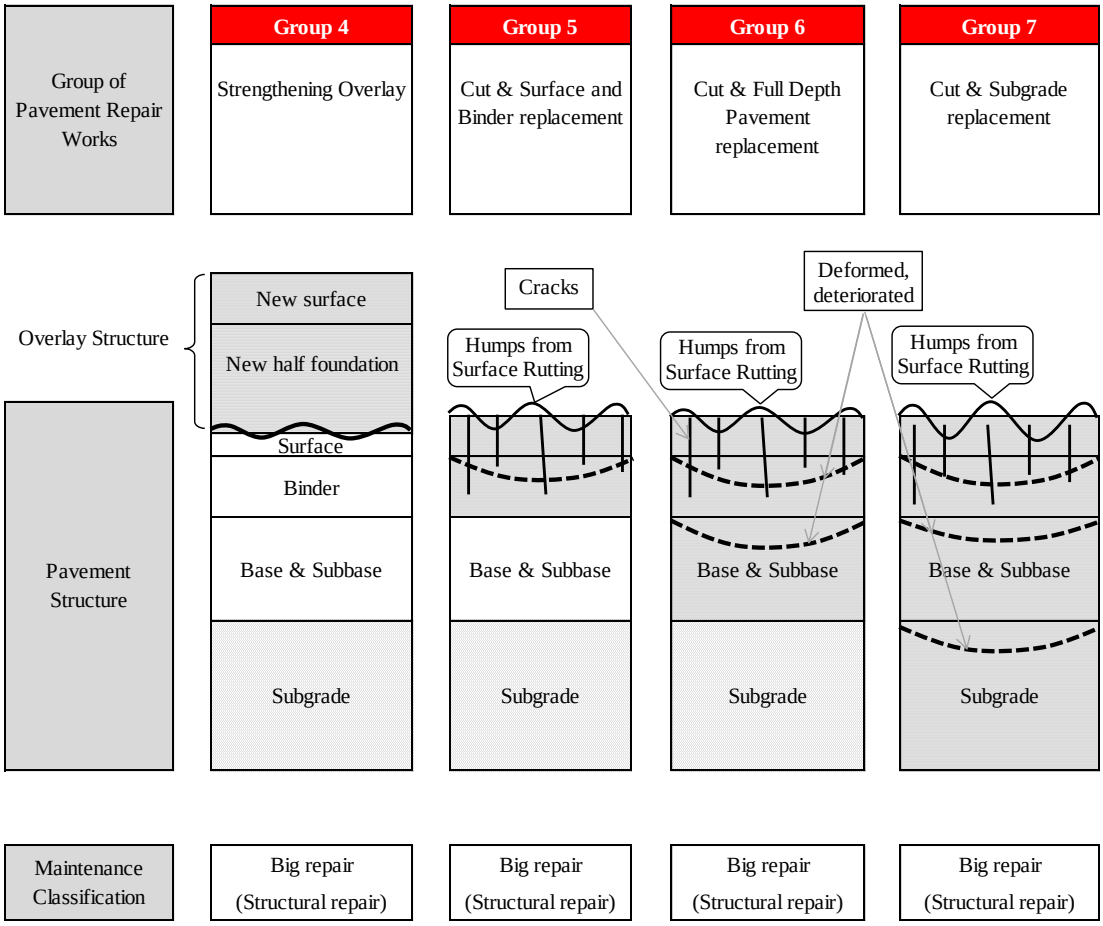
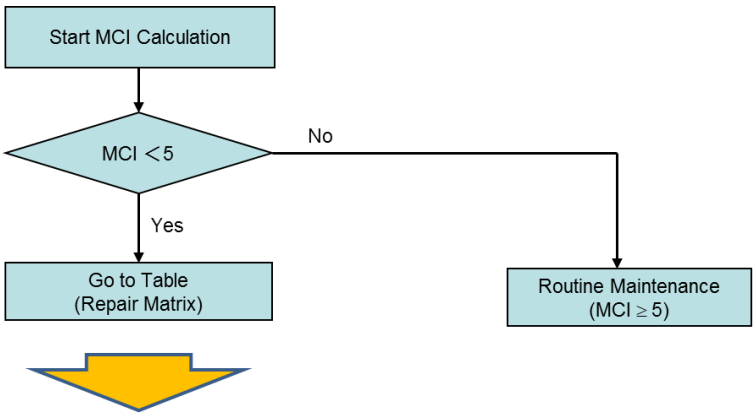


Figure 5.28 Common repair works for pavements in Vietnam (2/2)

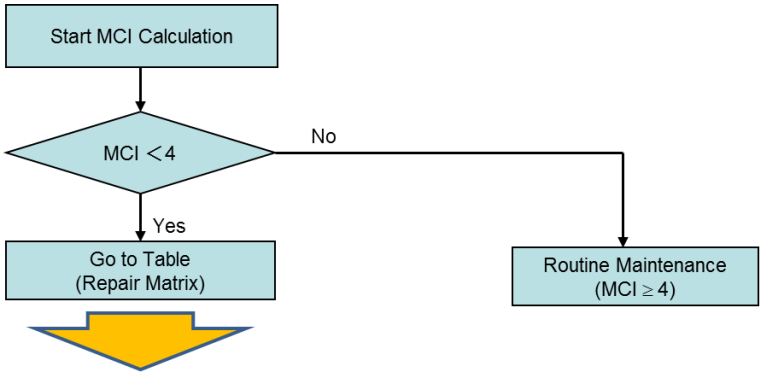
Within the practical project, pavement conditions have been surveyed and pavement condition data are available comprising of individual pavement indicators of IRI, rutting depth, cracking ratio. MCI value is also calculated based on available data. These available data are the basement for development of flowchart of pavement work repair selection in Vietnam.

Data collected in pavement condition survey in 2012 were analyzed to find out the relation between subjective selected repair works and pavement conditions. Flowchart of pavement work repair selection had been proposed referencing to Japanese practices in both the flow and selection of pavement index of MCI and individual indicators: IRI, rutting, and cracking ratio. Annual pavement maintenance plans that comprised of repair candidate sections and corresponding treatments were formulated by current method of engineer's judgement and application of the proposed flowchart. The proposed flowchart was modified based on comparison the two maintenance plans including taking into account of customization requirements that fit to domestic conditions. The flowcharts were developed for separated application to two groups of road classes (Figure 5.29, Figure 5.30).



Rutting Depth (RD, mm)			Light defects				Medium defects				Heavy defects			
			RD < 25 mm				[25 ; 40)				RD ≥ 40mm			
Traffic Volume (Heavy Vehicle: TV)			TV < 100	[100 ; 250)	[250 ; 1000)	TV ≥ 1000	TV < 100	[100 ; 250)	[250 ; 1000)	TV ≥ 1000	TV < 100	[100 ; 250)	[250 ; 1000)	TV ≥ 1000
Crack rate (CR), %	Light defects	CR < 5%	(Zone A) No repair				No repair			Cut & OL 50	OL 30	OL 30	OL 50	Cut & OL 50
		[5 ;15)	No repair		Surface treatment		OL 30	OL 30	OL 50	Cut & OL 70	OL 50	OL 50	OL 50	Cut & OL 70
	Medium defects	[15 ; 35)	OL 30	OL 50	OL 50	Cut & OL 70	OL 50	OL 50	OL 70	Cut & OL 70	OL 50	OL 50	OL 70	Cut & OL 70
	Heavy defects	[35 ; 50)	Cut & OL 50	Cut & OL 50	Cut & OL 70		Cut & OL 50	Cut & OL 50	Cut & OL 70		Cut & OL 50	Cut & OL 50	Cut & OL 70	
		CR ≥ 50%	(1) Surface & Binder replacement (2) Pavement whole layer replacement (3) Subgrade replacement (4) Strengthening Overlay (Zone C)				(1) Surface & Binder replacement (2) Pavement whole layer replacement (3) Subgrade replacement (4) Strengthening Overlay (Zone C)				(1) Surface & Binder replacement (2) Pavement whole layer replacement (3) Subgrade replacement (4) Strengthening Overlay (Zone C)			

Figure 5.29 Flowchart of Pavement Work Repair Selection in Vietnam - for Road class I, II, III



Traffic Volume (Heavy Vehicle: TV)			TV < 100	[100 ; 250)	[250 ; 1000)	TV ≥ 1000
Crack rate (CR), %	Light defects	CR < 5%	(Zone A) No repair or Daily maintenance (Crack seal, spot replacement etc.)			
		[5 ;15)				
	Medium defects	[15 ; 35)	OL 30 mm	OL 30 mm	OL 30 mm	
	Heavy defects	[35 ; 50)	(1) Pavement whole layer replacement (Big repair) (2) Strengthening Overlay (Zone C)			
		CR ≥ 50%				

Figure 5.30 Flowchart of Pavement Work Repair Selection in Vietnam - for other Road classes

In the matrices, pavement maintenance works are decided by its crack condition and rutting depth including consideration of traffic intensity. There are three zones in the matrices: (i) Zone A shows routine maintenance that is applied for pavements in good condition, (ii) Zone C shows the heaviest maintenance requirement for pavements in the most critical condition, (iii) Zone B is the remaining parts of the matrices that describes medium or non-structural repairs for pavements in fair condition with the focus on restoration its surface performance.

There are several types of structural repairs for pavements in Zone C: structural strengthening overlay and pavement replacement for surface course, full structure and deep replacement including subgrade treatment. To identify the proper measure, structural design must be implemented that requires for additional investigation of pavement bearing capacity or existing strength.

5.4.3 Master Database Operation

In order to ensure operation efficiency of system modules, the concept of master database (MASTER_DB) is applied. Structure of the master database is shown in Figure 5.31 that consists of five zones from Zone-A to Zone-E as shown below. Information to be stored in the master database is shown under the title of each zone in the figure.

- **Zone-A**: This zone preserves the system login information which includes list of user IDs, corresponding passwords and administrative rights. Those user IDs with administrative rights as “TRUE” has full rights to access and modify PMS master data as well. In contrast, those user IDs with administrative right as “FALSE” only has limited rights and cannot access and modify PMS master data.
- **Zone-B**: This zone saves an original PMS dataset developed separately by conversion software. Upon updating or importing PMS dataset into the PMS software, PMS dataset will be stored in this zone.
- **Zone-C**: This zone stores master data (pre-set condition data) that is inputted or modified by administrative users through system interfaces during software operation. It stores data of pavement deterioration ranking, repair work type and unit cost, repair policy, etc.
- **Zone-D**: This zone saves internally produced data including datasets generated from the original PMS dataset for the computation of each module. Three types of module datasets, namely pavement deterioration evaluation dataset, budget planning dataset and repair work planning dataset are stored in this zone. Also, Markov transition probabilities and pavement deterioration speed relative values, both of which are produced by Pavement Deterioration Evaluation Module.
- **Zone-E**: This zone stores partial results of pavement deterioration evaluation module. Data in this zone includes estimated hazard parameters and simulated Markov

transition probability matrix for individual pavement indicators (i.e. crack ratio, IRI, rutting depth).

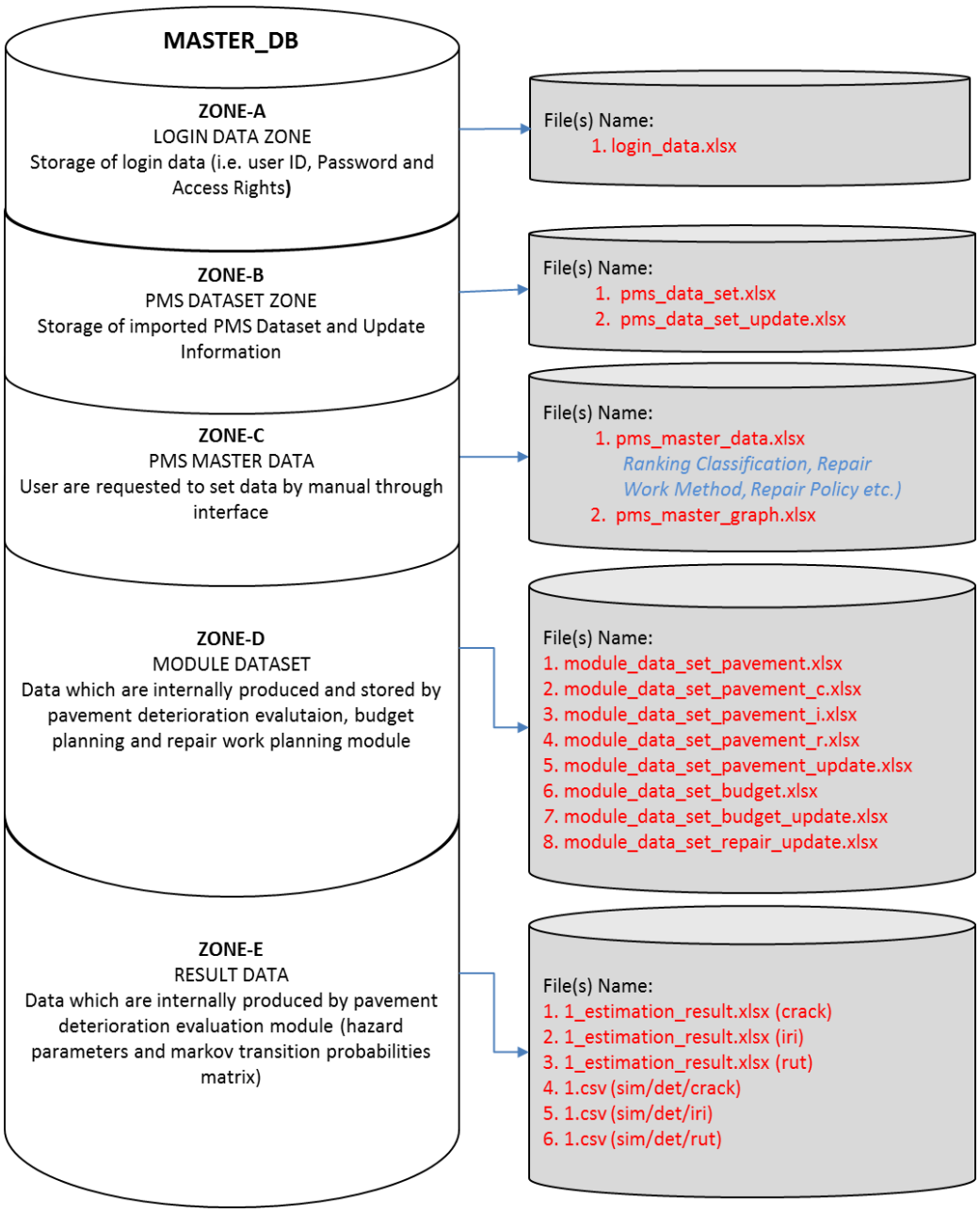


Figure 5.31 Master Database Configuration

5.4.4 Computation Flow of PMS System

5.4.4.1 General Computation Flow

The general flow of computation is shown in Figure 5.32**.** Step-by-step explanation is also shown, following the flowchart.

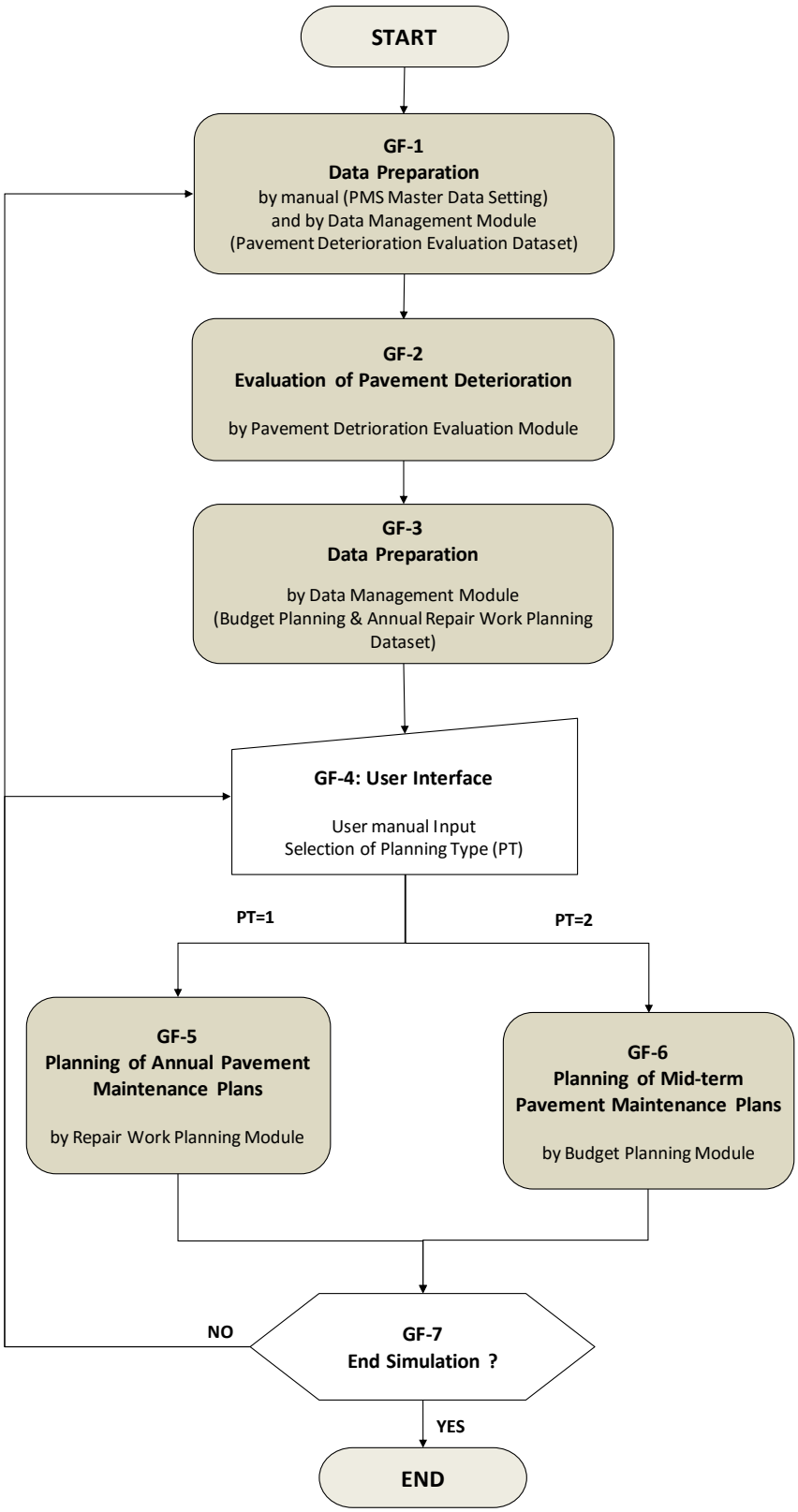


Figure 5.32 General Computation Flow

(1) GF-1:

Firstly in operation, Data Management Module imports and updates PMS datasets. Before generating pavement deterioration evaluation dataset, user shall confirm PMS master data if preset conditions are different from the default setting. Data Management Module formulates pavement deterioration evaluation datasets needed for the use of pavement deterioration evaluation based on the imported PMS datasets and PMS master data. The PMS datasets need to be developed separately prior to the importing into this planning software. Imported PMS datasets, PMS master data and created datasets for pavement deterioration module are saved in the MASTER_DB into Zone-B, Zone-C and Zone-D respectively.

(2) GF-2:

In the second step, the flow moves to Pavement Deterioration Evaluation Module. The main function of this module is to conduct a factor analysis in order to find out the most influential factors to the pavement deterioration. Then, the module goes into the analysis of pavement deterioration speed based on Markov transition probability theory. Benchmarking evaluation is also performed in this stage by using Pavement Deterioration Evaluation Module. Some internally generated data are stored in Zone E. The final outputs of hazard parameter, Markov transition probabilities, and benchmarking evaluation results are saved in different output folders form MASTER_DB.

(3) GF-3:

In the third step, the flow goes to Data Management Module and generates budget planning and annual repair work planning dataset using PMS dataset, PMS master data and result of pavement deterioration evaluation module (particularly benchmarking information). The outputs (i.e. budget planning module dataset and annual repair work planning dataset) are stored in Zone-D of the MASTER_DB.

(4) GF-4:

Users are requested to select planning types, either (1) Annual Pavement Repair Plans or (2) Middle-term Pavement Repair Plans.

(5) GF-5 and GF-6:

Based on the selected planning type, the software goes into either one of these modules; (1) Repair Work Planning Module for annual pavement repair plans, or (2) Budget Planning Module for middle-term pavement repair plans.

In addition, users are requested to input manually conditional data of simulation follow the guidance given by user interfaces.

(6) GF-7:

If simulation repetition is necessary, user can return either at GF-1 or GF-4. If master data need to be revised, user shall return to GF-1 and then proceed to remaining steps. Similarly, if repair policy or simulation condition or budget condition or all of them need to be revised, user shall return to GF-4 and then proceed to remaining steps.

5.4.4.2 Data Management Module

Main function of Data Management Module is to import and update PMS dataset into the system, select data items from the imported PMS dataset and internally generated data, and classify and divide them into independent datasets needed for the computation of three system modules.

The operation flow of Data Management Module is illustrated in Figure 5.33. Step-by-step explanation is also described, following the flowchart.

(1) DM-1: Import PMS dataset

In the first step, Data Management Module imports and updates PMS dataset and save it in Zone -B of the MASTER_DB.

(2) DM-2: User Interface (Manual data input by user)

Pre-set condition data commonly used in all calculation modules are required to be set in this step, which consist of pavement deterioration rankings or condition states, pavement repair policy, repair work selection criteria, and repair work unit costs. The inputted data are saved in Zone-C of the MASTER_DB.

(3) C-1: Pavement deterioration rankings or condition states

Pavement deterioration rank shows the condition state of pavement within the range from the best condition to the worst condition. Rankings can be defined for all three pavement deterioration indicators: (i) crack ratio, (ii) rutting depth and (iii) IRI value. Table 5.18 shows a data input form which saves inputted data in MASTER_DB after user's manual data inputting.

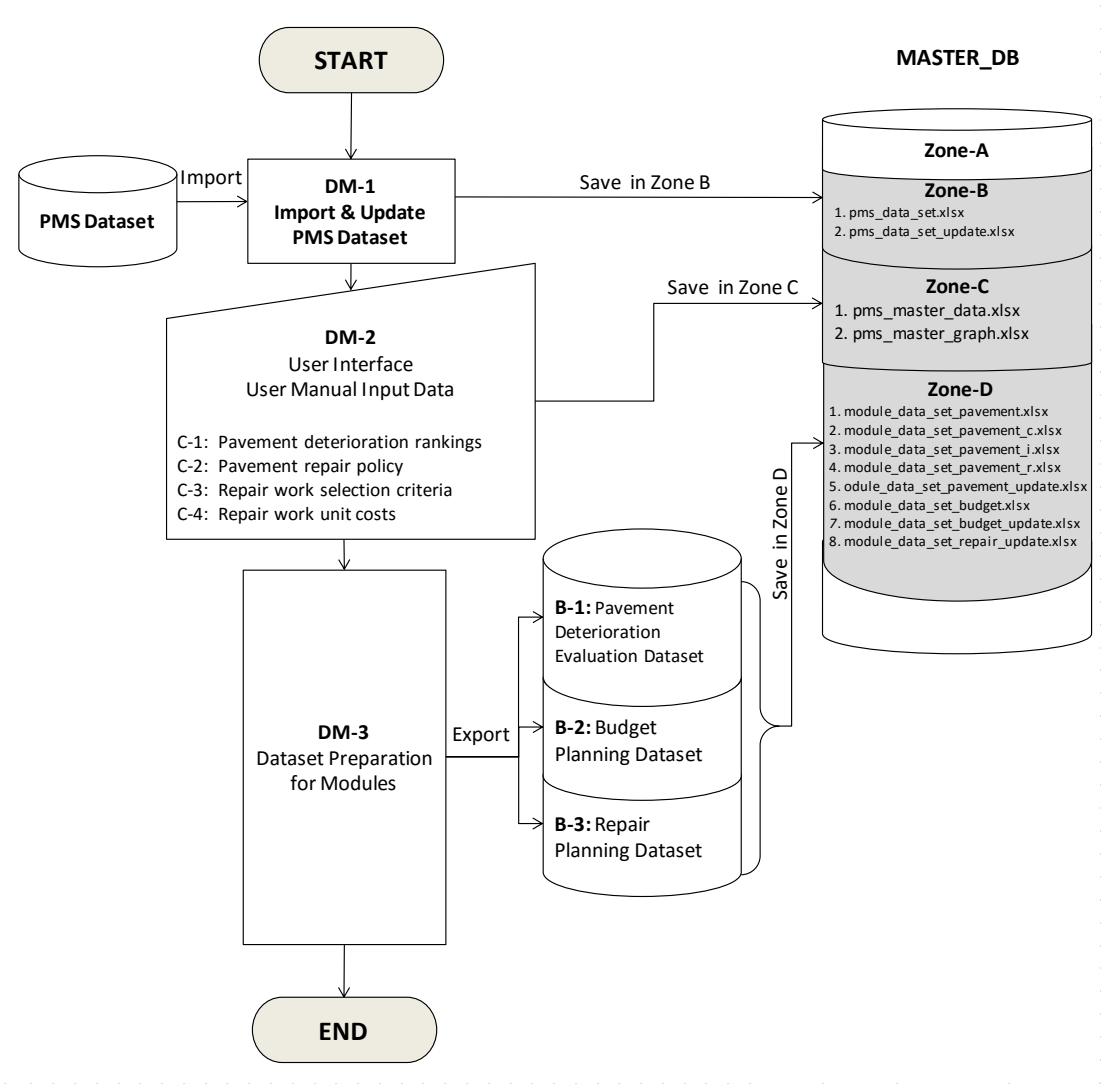


Figure 5.33 Flowchart - Data Management Module

Table 5.18 Data Input Form for Definition of Pavement Condition States

Rank	Tab-1: Crack Ratio (%)			Tab-2: Rut Depth (mm)			Tab-3: IRI (mm/m)		
1		≦CR <			≦RD <			≦IRI <	
2		≦CR <			≦RD <			≦IRI <	
3		≦CR <			≦RD <			≦IRI <	
4		≦CR <			≦RD <			≦IRI <	
5		≦CR <			≦RD <			≦IRI <	
6		≦CR <			≦RD <			≦IRI <	
7		≦CR <			≦RD <			≦IRI <	
8		≦CR <			≦RD <			≦IRI <	
9		≦CR <			≦RD <			≦IRI <	
10		≦CR <			≦RD <			≦IRI <	

(4) C-2: Pavement repair policy

Pavement repair policy is defined by indicating the condition states of pavement which need being repaired with corresponding repair type that is shown by colors. Users are requested to select candidate repair ranks for each type of pavement deterioration and setting the corresponding repair works by selection proper color indicators. Table 5.19 shows one example of pavement repair policy setting for the case of six condition states from 1 to 6; two types of repair works are proposed: (i) cut & overlay must be applied in the case of pavement rank is 6 for crack and roughness deterioration, and pavement rank is 6 or 5 for rutting deterioration; (ii) cut & replacement must be applied in the case of pavement rank is 5 for rutting deterioration, and pavement rank is 5 or 4 for crack and roughness deterioration.

Table 5.19 One Sample of Repair Policy

Rank	Crack Ratio	Rutting Depth	IRI
1			
2			
3			
4			
5			
6			

Note) Explanation of color indicators

Color Indicator	Description of repair method
	Cut & Overlay (Non-Structural)
	Cut & Replacement (Structural)

(5) C-4: Repair work unit costs

Data of unit costs for repair works are needed for maintenance budget estimation. Cost estimation is available in both annual maintenance plan and mid-term maintenance plan. All typical repair work types which are practically applied in Vietnam are integrated to meet the demand of customization to domestic conditions. Same values of repair costs are applied for both repair cost computations. Table 5.20 shows a data input form for pavement repair unit costs which saves inputted data in MASTER_DB after user's manual data inputting.

Table 5.20 Repair Work Unit Costs

No.	Repair Work Items	Unit	Unit Cost (1000 VND)	Remarks
1	**Surface treatment**			
	1.1 Crack Seal	m2		
	1.2 Seal Coat	m2		
	1.3 Slurry Seal	m2		
	1.4 Patching with Hot Asphalt Mixture	m2		
	1.5 Patching with Cold Asphalt Mixture	m2		
	1.6 Pavement Partial Replacement	m2		
	1.7 Surface Level Difference Elimination	m2		
2	**Thin overlay**			
	2.1 Overlay (30 mm)	m2		
	2.2 Overlay (50 mm)	m2		
	2.3 Overlay (70 mm)	m2		
	2.4 SBST, DBST, TBST			
	2.5 Bituminous penetrated macadam			
3	**Cut and overlay**			
	3.1 Cut & Overlay (30 mm)	m2		
	3.2 Cut & Overlay (50 mm)	m2		
	3.3 Cut & Overlay (70 mm)	m2		
4	**Surface and Binder Replacement** (for standard type)	m2		
5	**Whole Layer Replacement** (for standard type)	m2		
6	**Subgrade Replacement** (for standard type)	m2		
7	**Strengthening Overlay**	m2		

(6) DM-3: Dataset preparation for modules

Based on the imported PMS datasets, three individual datasets are prepared for the operation of three modules shown below and saved in the Zone-D of the MASTER_DB. A list of data requirements for each dataset needs to be prepared and saved in advance in Zone-B of the MASTER_DB.

5.4.4.3 Pavement Deterioration Evaluation Module

The operation flow of Pavement Deterioration Evaluation Module is illustrated in Figure 5.34. Step-by-step explanation is also shown, following the flowchart.

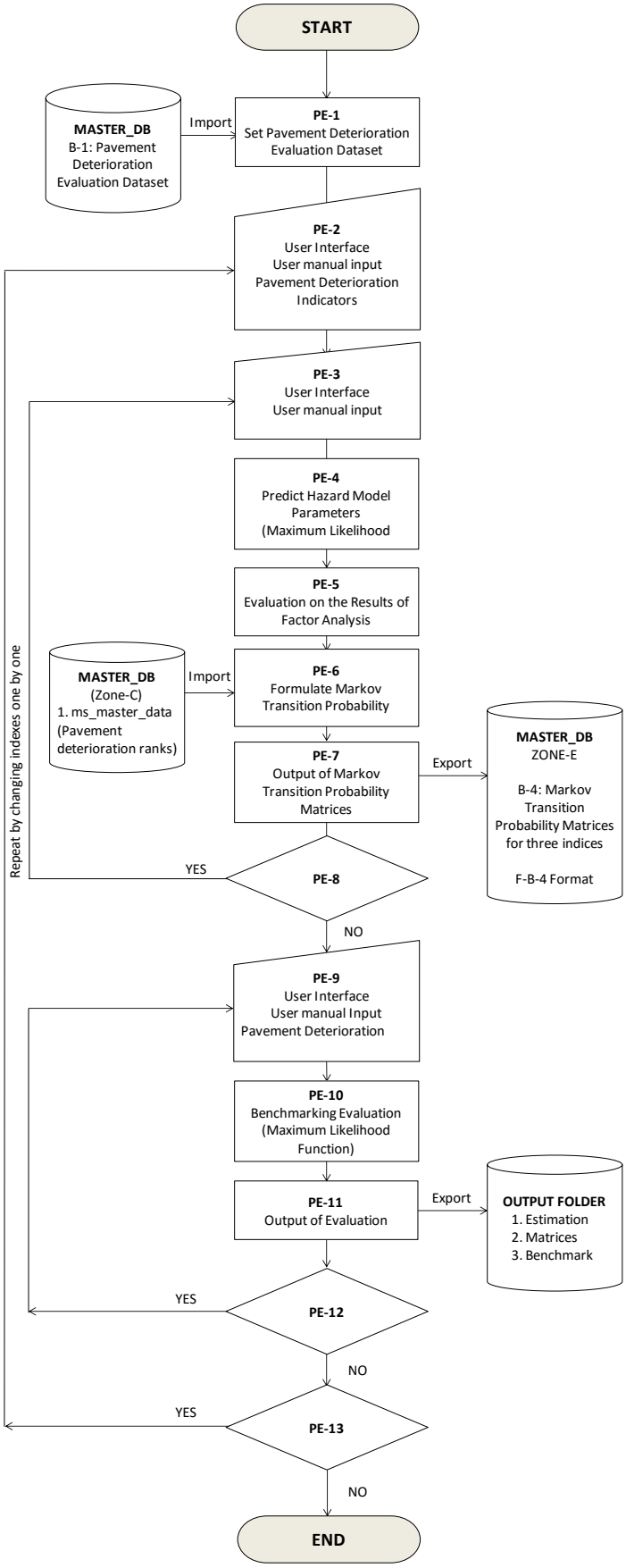


Figure 5.34 Flowchart - Pavement Deterioration Evaluation Module

(1) PE-1: Import pavement deterioration evaluation dataset

In the first step, Pavement Deterioration Evaluation Module imports the dataset for this module from the Master Database that was already generated in Data Management Module and saved in MASTER_DB.

(2) PE-2: Set pavement deterioration ranks through user interface (User manual input)

Users are requested to select each pavement deterioration indicator for calculation in next steps. Hereafter, simulation will be executed for all steps from PE-2 to PE-13 for the selected pavement deterioration indicator until the completion for all indicators.

(3) PE-3: Set factors for factor analysis through user interface (Use manual input)

Factor analysis is executed to find out the most influential factors to pavement deterioration based on the latest pavement condition survey data. Naturally, there are various factors that make contribution much or less to pavement deterioration. Users can specify maximum up to four factors at a time from the set of factors. Factor analysis will be executed for all steps from PE-3 to PE-8 for the specified factors until completion of factor selection.

(4) PE-4: Predict hazard model parameters

Then, hazard parameters will be computed based on the selected factors. A total of seven (7) factors are considered for factorial analysis. Up to four (4) factors can be selected at a time of analysis. A hazard parameter is defined as a probability of pavement deterioration changing from one rank to the next rank in terms of deterioration progress. An inverse value of this hazard parameter indicates an expected life expectancy of the pavement for the rank under consideration**.**

(5) PE-5: Evaluation on the results of factor analysis

Evaluation of calculated hazard parameters is conducted, applying t-tests in statistical theory to determine parameters. Based on t-test results, users can understand how the selected factors take influence to the deterioration of pavement**.**

(6) PE-6/PE-7: Formulate Markov transition probability matrices and store in MASTER_DB

Based on the hazard parameters computed in the previous steps, Markov transition probabilities can be calculated for each type of pavement deterioration like: cracking, rutting, and roughness.

Computed probabilities are all saved in the MASTER_DB. Figure 5.35 shows the concept of Markov transition probabilities. Markov transition probability is defined as time-series probabilities of pavement deterioration changing from one rank to other ranks or keeping remaining in current rank as shown in the figure below. The module computes the probabilities by Markov hazard model using at least two time-series datasets of pavement conditions.

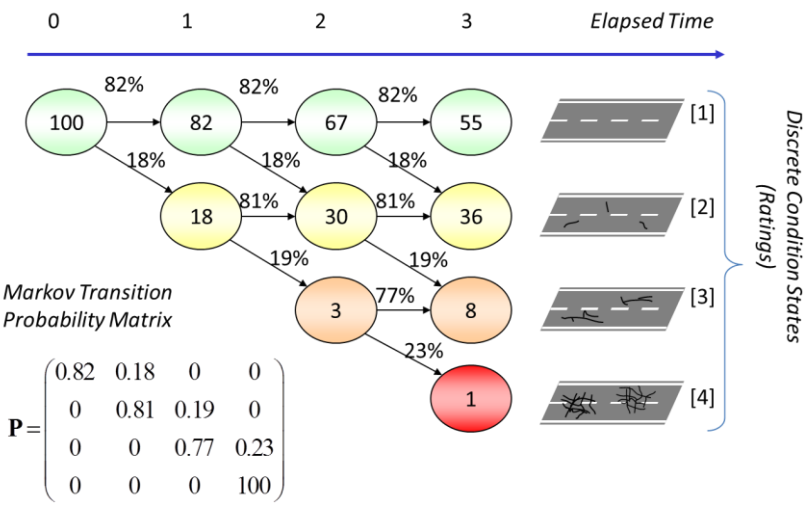


Figure 5.35 Concept of Markov Transition Probabilities

Markov hazard model is the main algorithm for these calculations including estimation pavement deterioration hazard rates that can be used to determine life expectancy in each ranking and in whole.

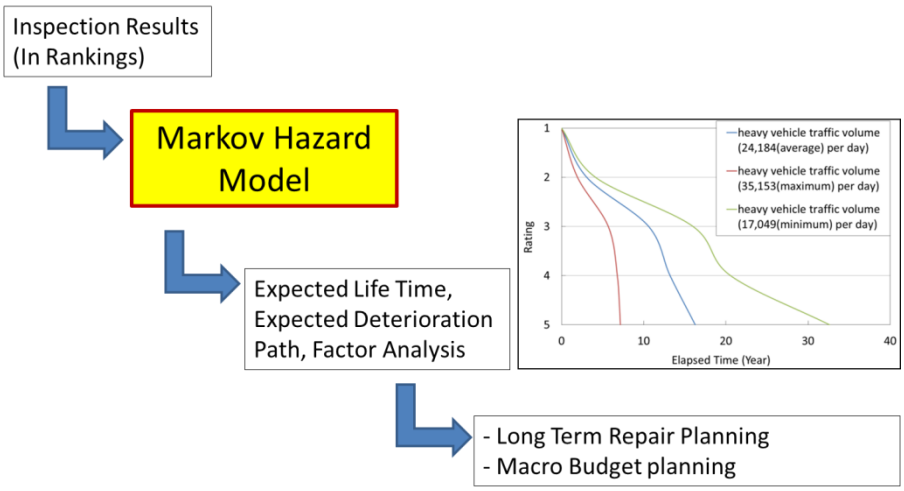


Figure 5.36 Markov Hazard Model for Pavement Deterioration Forecasting in whole

(7) PE-3 – PE-8: Repeat simulation for all factors

(8) PE-9: Set pavement deterioration evaluation unit through user interface

Users will be requested to input inventory parameters for grouping all road pavement sections into groups for executing Benchmarking evaluation in the next step. There are many inventory parameters can be considered in this case depends on the availability of data and users' ideas or judgment. If road number is selected as inventory parameter for grouping, sub-sets of data for each group of sections that belongs to same road can be prepared. Similarly, it can be applied for many other groups with the same group identifier such as: road regional administrator or maintenance contractors, traffic intensity, road class, road geometric type, climate condition, pavement structure, road section type, pavement lane type, embankment condition, pavement technology, etc.,.

(9) PE-10: Benchmarking evaluation of pavement deterioration speeds

In this step, further analysis is made on the variance of pavement deterioration speeds in the form of relative speed from an average deterioration speed by incorporation of benchmarking analysis applying local mixture Markov hazard model for different groups of pavement sections. There is also deep discussion of Benchmarking methodology in Chapter 2 and its full application.

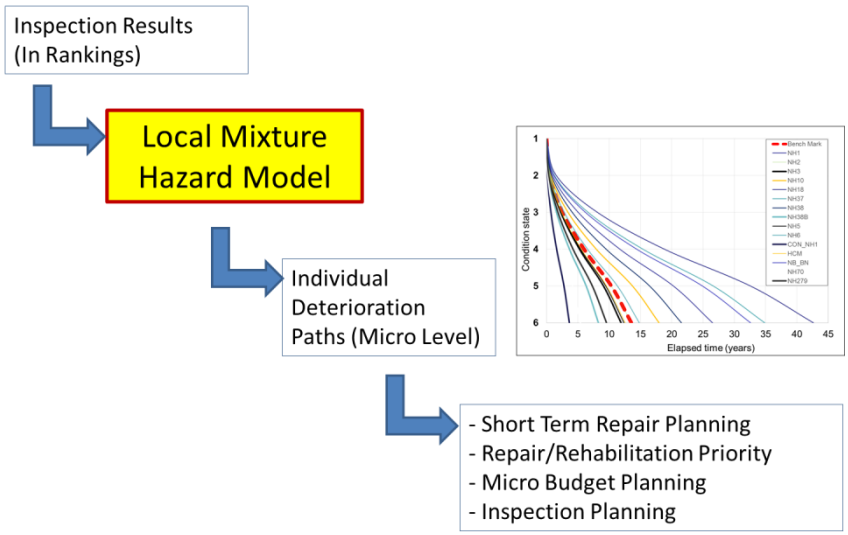


Figure 5.37 Mixed Markov Hazard Model for Pavement Deterioration Forecasting in whole

Based on the imported dataset (B-1), parameters are computed that show relative speeds (ε) of pavement deterioration for each group of road sections. Computation results of the pavement deterioration speeds for all three pavement deterioration indicators are all saved in the MASTER_DB. Comparison relative speeds of pavement deterioration, road administrators would have a brighter image about pavement deterioration in the road network under their jurisdiction. Specified abnormal cases with faster deterioration

speeds will be targeted for further study or survey especially on pavement bearing capacity for finding out the causes or taking right actions to improve the situation.

(10) PE-8 – PE-11: Repeat simulation for all pavement deterioration evaluation units

In this step, user can repeat the simulation of other pavement deterioration unit and can evaluate the pavement deterioration. The same process described between PE-8 and PE-10 will be repeated.

(11) PE-2 – PE-13: Repeat simulation for all pavement deterioration indexes

Simulation for other remaining pavement deterioration indices other than initially or already simulated indices can be repeated in the same procedure as described between PE-2 and PE-13.

5.4.4.4 Budget Planning Module

The operation flow of Budget Planning Module (medium-term budget planning) is illustrated in Figure 5.38. Step-by-step explanation is also presented, following the flowchart.

(1) BP-1: Import budget planning dataset

The first step of this Budget Planning Module is to import the budget planning module dataset for the computation of this module from the MASTER_DB.

(2) BP-2: Import pavement condition data

Pavement condition data, which include Markov transition probability matrices, pavement deterioration ranks and repair policy, are imported from Master_DB.

(3) BP-3: Update repair work unit cost

In general, repair unit cost is subject to change with market price of labor and material. Therefore, it is necessary to update the repair work unit cost. Users are requested to input manually unit costs of repair work in the form for updating.

(4) BP-4: Set simulation conditions

Users are requested to input simulation information that consists of simulation time and the number of Monte Carlo simulation. Default values will be preliminary set for the number of Monte Carlo simulation.

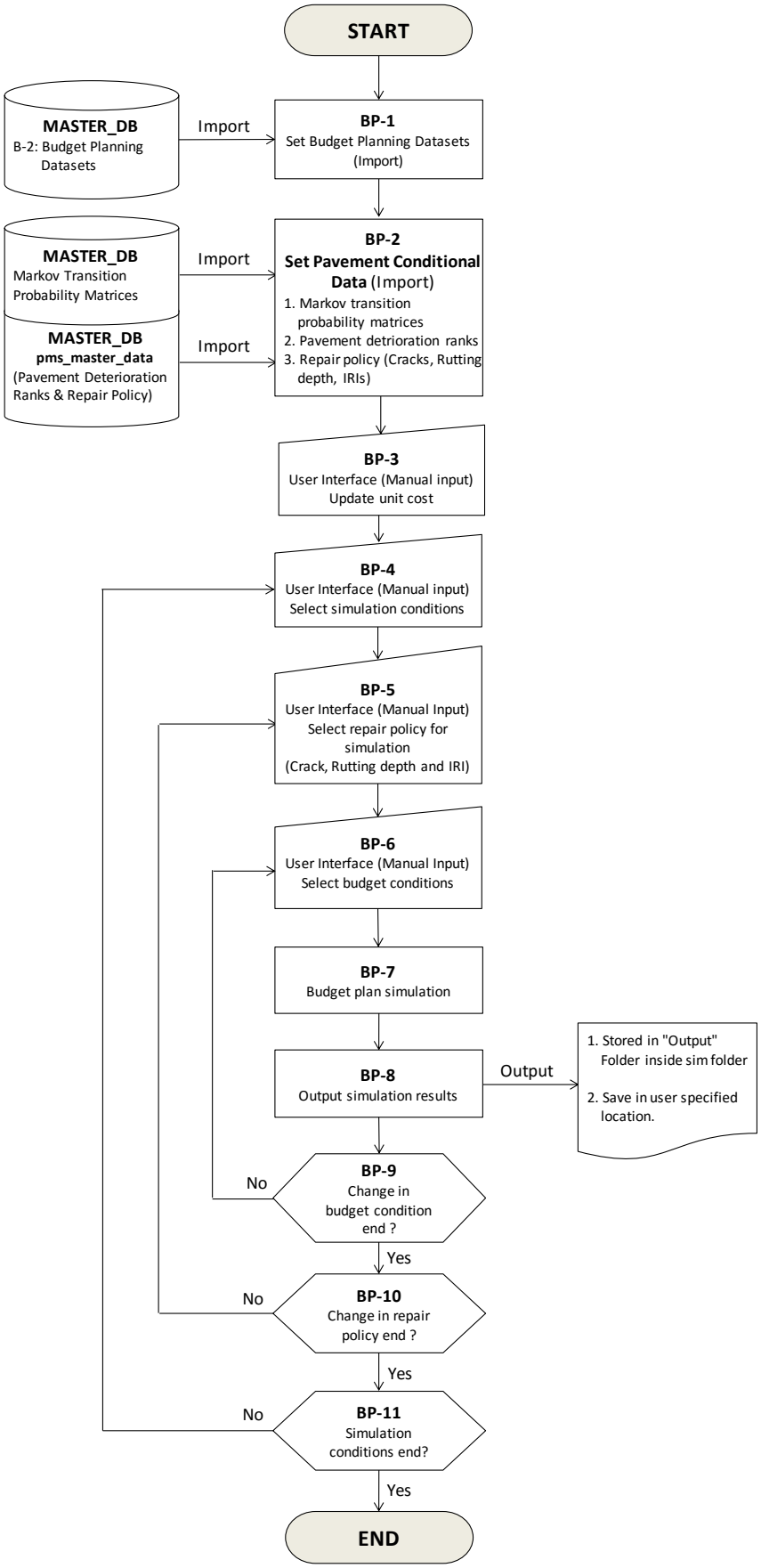


Figure 5.38 Flowchart - Budget Planning Module

(5) BP-5/BP-6: Select repair policy for simulation and select budget conditions

In this step, users are requested to input the simulation trial conditions. Users are allowed to set trial cases, changing the following conditions

- Change in repair policy conditions
- Existence of budget constraints
- Change in budget conditions

Simulation conditions can be set for three following scenarios of maintenance strategy

- **Scenario-1**: Remaining current budget level. Among three scenarios, this is the worst case of maintenance in term of resource allocation. Based on such constraint, progress of pavement deterioration when current budget level is maintained will be clarified.
- **Scenario-2**: Remaining current level of pavement conditions for the road network. Under the accelerated process of pavement deterioration, keep remaining the current distribution of pavement conditions on the whole road network to retard its deterioration progress is the maintenance strategy in this case. And requirement on corresponding amount of budget allocation must be determined.
- **Scenario-3**: There is a certain targeted level of services for pavement conditions of the whole road network. This strategy is the best case among three scenarios that also demands for bigger amount of budget for road maintenance.

(6) BP-7: Budget plan simulation

Upon completing the all settings required for budget planning, the module will simulate the budget by taking account of all setting condition by using Monte Carlo simulation.

(7) BP-8: Output and save data of simulation results

The outputs of the simulation results will be saved inside the system. Also, user can export the simulation result that consist of “Cost”, “Budget”, “Condition (Crack, Rutting, and IRI)”, and “Maintenance Risk” will be saved inside the system. Also, users can export the simulation result in users’ specified folder in MS-Excel format in both value sheets and charts.

The other groups of steps: BP-6 to BP-9, BP-5 to BP-10, BP-4 to BP-11 can be selected for repeating simulation in case of changing budget constraints, repair policy, and simulation conditions, respectively.

(8) Making the List of Repair Candidate Sections in Mid-term Plans

It is commonly understood that one of the most key information in pavement mid-term plans is the budget proposal. In Vietnam, pursuant to the Law on State Budget, there must be work volume in road mid-term plans that is used to calculate budget requirement.

Therefore, preparation of maintenance plans for individual year in the planned mid-term must be implemented. Calculation flow is shown in Figure 5.39.

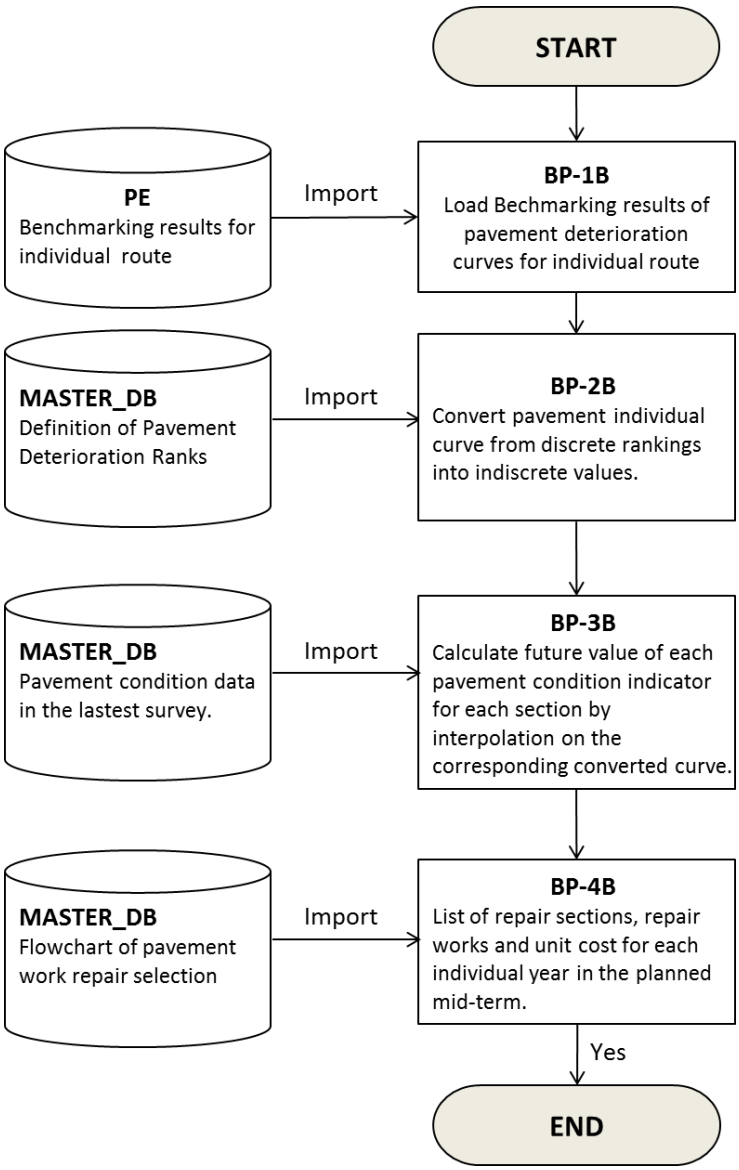


Figure 5.39 Calculation Flow of Making the List of Pavement Repair for Individual Year in the Planned Mid-term

Given the real crack ratio Cr^{i}_{t1} of section (i) at the surveyed time t_1 *(ex, 2012)*, future crack ratio Cr^{i}_{t2} of the pavement section (i) at time t_2 after Δt years from t_1 can be determined by

simple projection on the converted pavement deterioration curve as shown in the illustration in Figure 5.40.

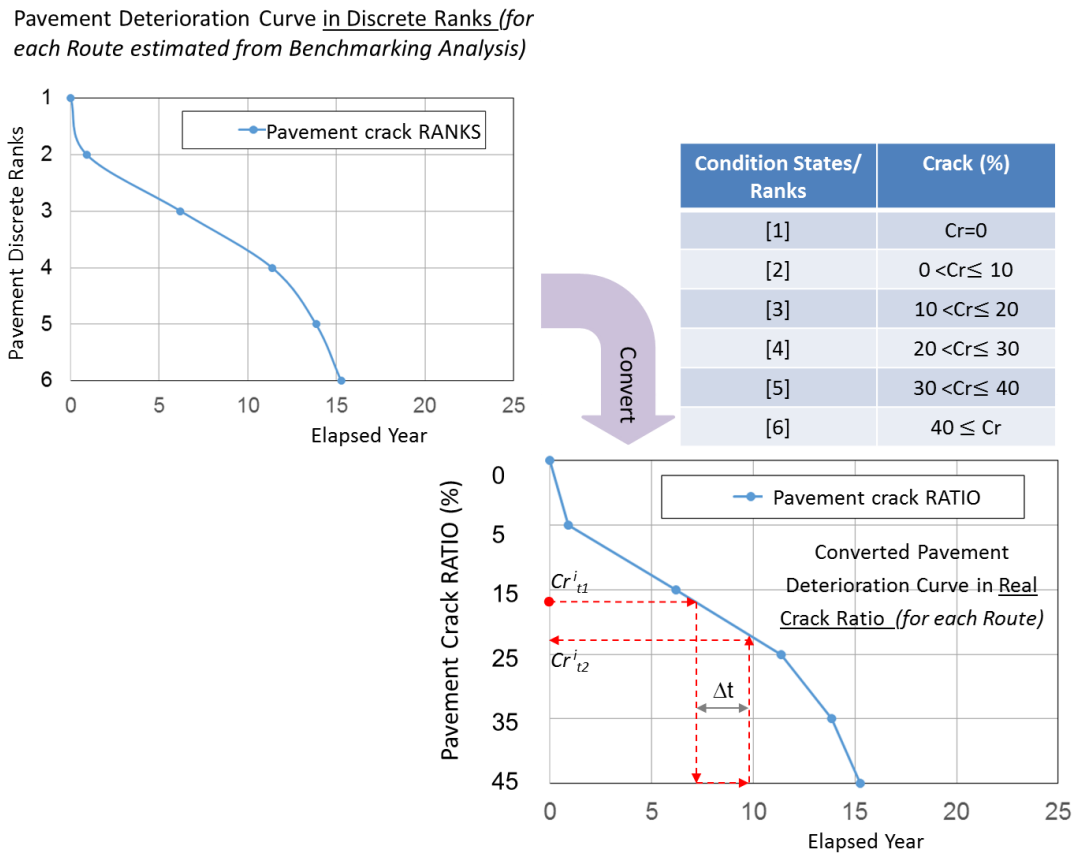


Figure 5.40 Determination Future Condition States for Individual Pavement Section

5.4.4.5 Repair Work Planning Module (Annual Planning)

The flowchart of the Repair Work Planning Module (Annual planning) is illustrated in Figure 5.41. Step-by-step explanation is also shown, following the figure.

(1) RP-1: Import the repair work planning dataset (B-3)

In the first step, Repair Work Planning Module imports the repair planning dataset from the MASTER_DB.

(2) RP-2: Check pavement type

Since repair work types are remarkably different by pavement type (asphalt concrete, bitumen pavement, and cement concrete) this module check the pavement type at this stage and proceed further for the remaining steps. However, with the majority of asphalt concrete pavement on national road network, the default parameters such as flowchart of work repair selection in other next steps are only applicable for this type of pavement.

Without default parameters, manual setting can be applied for other remaining pavement types.

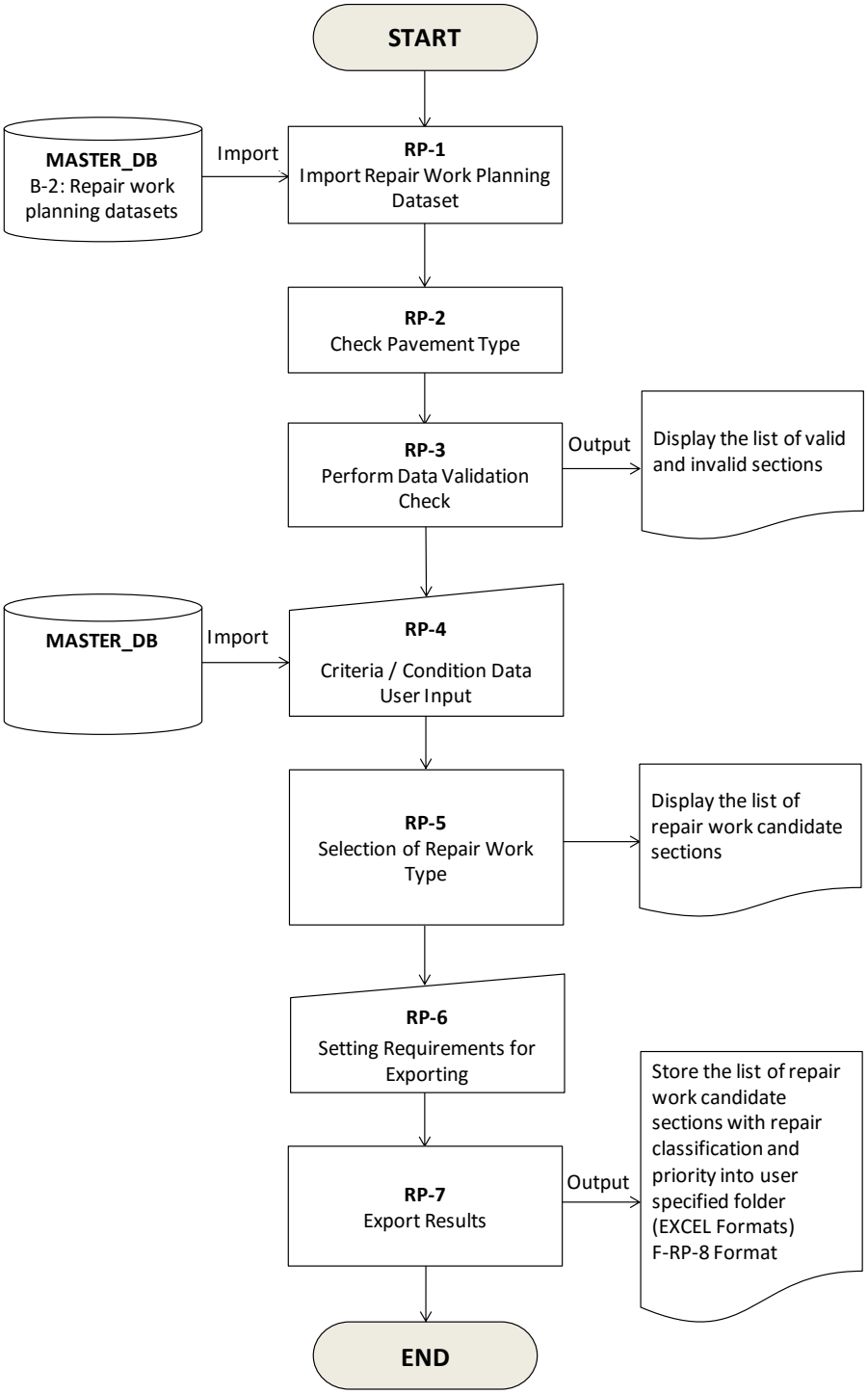


Figure 5.41 Flowchart - Repair Work Planning Module

(3) RP-3: Perform data validation check

The validation check for imported repair work planning module dataset is performed to check whether the road sections and values are proper in accordance with definition of

each data items. If any of the road section or data of particular sections are improper, those sections or values are visualized by different color. Improper sections and values are excluded from the analysis from hereafter.

(4) RP-4: Criteria / Condition data (user manual input)

Before running the module for identifying appropriate repair work type, conditions or criteria including unit costs shall be defined at this stage. The default value will be imported automatically from MASTER_DB. It is also possible for users to modify conditions for their new definition.

(5) RP-5: Selection of repair work type

In this step, the module analyzes data (i.e. road class, crack rate, rutting depth and traffic volume) to suggest proper repair work for each pavement section based on the flowchart of work repair selection (in Section 5.4.2.2). Users can modify the flowchart for whole application during module operation or update repair type for individual section including corresponding unit cost in the output sheets. The results of repair work planning module are categorized by repair method and repair classification. Repair costs for routine maintenance and medium repairs (Zone A and Zone B in the flowchart of work repair selection) are calculated automatically. However, in case of big repairs in the Zone C, list of corresponding repair sections is made by the module but further activities of structural investigation or testing, pavement design and cost estimation must be conducted out of the PMS system to complete pavement annual maintenance plans.

(6) RP-6: Export condition data (user manual input)

Results of the module can be exported in various formats such as by road name, direction, and identified sections (From / To) for the following categories;

- All sections
- Out of analysis sections
- Target repair sections
- No or minor repair sections
- Medium repair sections
- Big repair sections

Priority of repair is also set in this stage by taking MCI value into account. Sections with the lowest MCI value shall get the highest priority for repair work under each repair work classification. All the candidate repair work sections are listed in increasing order of MCI value. In principle, section listed up at the top shall get the first priority for repair work.

For more accurate maintenance plans that can be put into implementation, it is highly recommended to cross-verify the result with actual field condition to made confirmation or modification, updating in proper cases. If actual field conditions differ from the analysis result, adjustment in the priority list shall be made.

5.4.5 Data Requirements and PMS Dataset Formulation and Structure

Proper data items are requested for operation of each system module. These data items can be extracted from imported PMS dataset or generated by data management module. Requested data items for each module are shown in Table 5.21 that creates overall requirement for PMS data set or its structure.

There are four categories of data in PMS dataset that comprise of: (i) road asset data, (ii) pavement condition data, (iii) repair history data, and (iv) traffic volume data. Among fifty data items, most of them are used for repair work planning module or planning at project level to formulate annual maintenance plans including detail information of individual candidate repair section such as: its location, current conditions, repair method, unit cost, priority, et cetera. In case of planning at network level, both pavement deterioration evaluation module and budget planning module demand for very few data items among these listed in Table 5.21. That's the remarkable advantage of PMS Kyoto model in comparison with HDM-4 with the demand for 159 data items (SAPI-2) [2].

As the above description, to estimate pavement deterioration curves by stochastic prediction model, at least two time-series datasets of pavement condition must be prepared. Within project framework, only one pavement condition survey was conducted in 2012 to collect the three main pavement indicators of IRI, crack ratio and rutting depth that leads to the problem of lacking one time-series pavement condition dataset. However, the situation was treated smoothly based on another advantage of PMS Kyoto model's methodology.

Table 5.21 Data Requirements for Modules

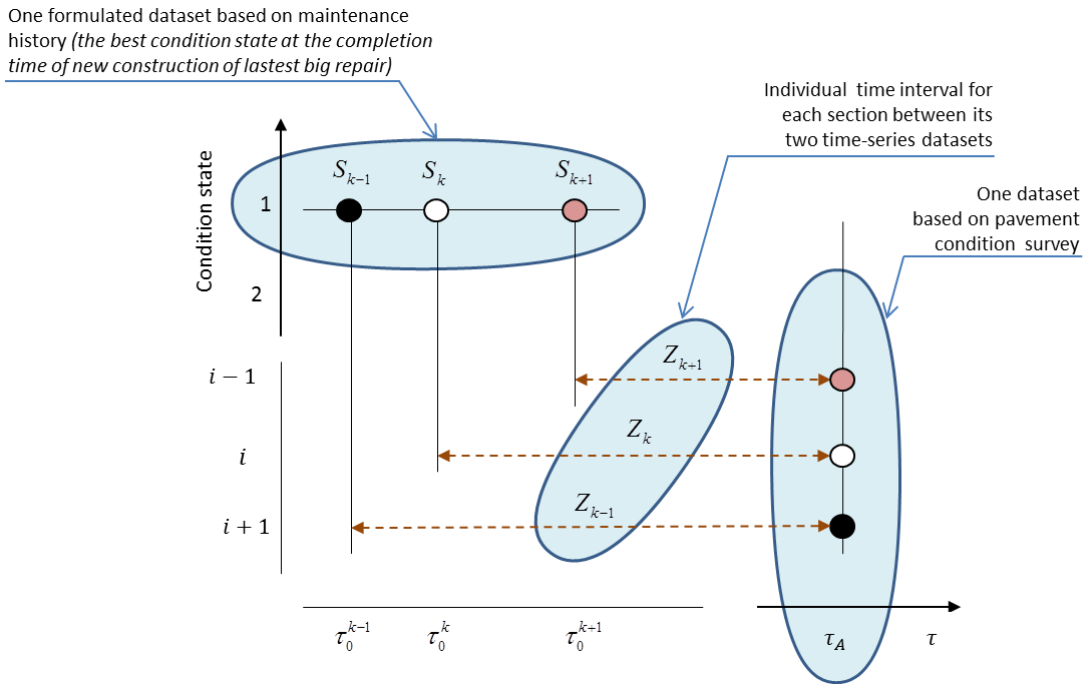
Data Category	No	Data Items			Unit	System Modules		
						Pavement Deterioration Evaluation Module	Repair Work Planning Module	Budget Planning Module
ROAD ASSET DATA	1	Road ID					x	
	2	Route Number					x	
	3	Road Name				*	x	
	4	Road Branch Number				*	x	
	5	Regional Road Administrator				*	x	
	6	Road Management Field Office					x	
	7	Structural Type				*		
	8	Geographical Area				*		
	9	Road Section	From	Km	km		x	
	10			M	m		x	
	11		To	Km	km		x	
	12			M	m		x	
	13	Section Length			m		x	x
	14	Number of Lanes				*	x	
	15	Up or Down					x	
	16	Pavement Type			m	+,*	x	x
	17	Pavement Width			m		x	x
	18	Pavement Thickness			cm	+		
	19	Climate	Annual Rainfall			+		
	20		Temperature			+		
	21	Topograph conditions	Flat / Rolling / Mountainous Terrain			+		
PAVEMENT CONDITION DATA	22	Latest Condition Survey	Year/month of survey			x	x	
	23		Position of Lane surveyed				x	
	24		Pavement Type				x	
	25		Crack Rate	Cracking	%		x	
	26			Patching	%		x	
	27			Pothole	%		x	
	28			Total	%	x	x	x
	29		Rut Depth	Max	mm		x	
	30			Average	mm	x	x	x
	31		IRI		m/km	x	x	x
	32	2nd Latest Condition Survey	Year/month of survey			x		
	33		Position of Lane surveyed					
	34		Pavement Type					
	35		Crack Rate	Cracking	%			
	36			Patching	%			
	37			Pothole	%			
	38			Total	%	x		
	39		Rut Depth	Max	mm			
	40			Average	mm	x		
	41		IRI		m/km	x		
	42	MCI					x	
REPAIR HISTORY DATA	43	Latest Repair	Year / Month of the latest repair or new construction			x	x	
	44		Repaired Lane				x	
	45		Repair Method			+, *	x	
	46		Repair Classification				x	
TRAFFIC VOLUME DATA	47	Latest survey	Total traffic volume		AADT			x
	48		Heavy traffic volume		AADT	+	x	x
	49	2nd Latest Survey	Total traffic volume		AADT	+		
	50		Heavy traffic volume		AADT	+		

Notes) x: Required data items

**: Specific identifier for Benchmarking analysis*

+: Items considered for factor analysis

Thanks to Kyoto model with many advanced analysis incorporated, it is possible to formulate another dataset just utilize pavement history data with the assumption to set the best condition state for pavement at the completion time of new construction or the latest big repair (Figure 5.42). PMS dataset has been prepared for pavement sections of 100 meters long for individual lane in each direction. That is accepted because of application of the discrete condition state concept in Kyoto model. Moreover, there is also no constraint of time interval between two time-series condition data for all sections in the model. It means that time interval can be different among sections and time interval between two surveys can be different also.



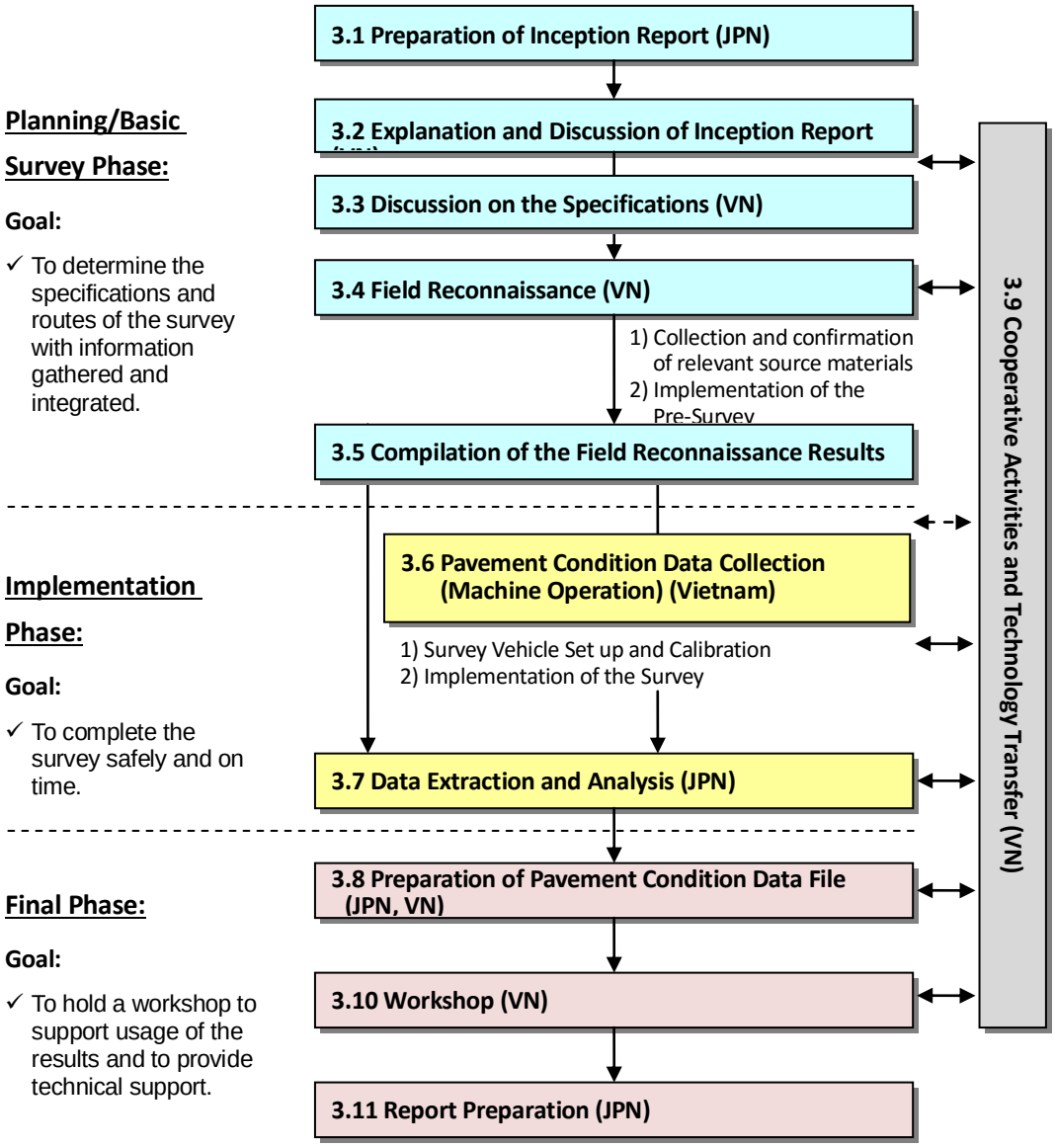
Note) In this example, sections S_{k-1}, S_k, S_{k+1} have best condition state (ranking 1) at their completion time of new construction or lastest big repair of τ_0^{k-1}, τ_0^k, τ_0^{k+1} respectively. Such time information is obtained from pavement maintenance history. Pavement condition survey had been conducted one time only at τ_A

Figure 5.42 Formulation Pavement Condition Dataset based on Maintenance History

5.4.6 Implementation of Pavement Condition Survey

Specialized team in pavement condition survey and data processing had been dispatched from Japan to work in Vietnam between February 2012 and March 2013 to collect data for national routes in the northern Vietnam with total length of 2,303 km corresponding to 4,606 lane-kilometers in both directions. Figure 5.43 shows the survey operation flow. Automatic technology for pavement data collection has been applied using automated survey vehicle at normal travel speeds to increase productivity and improve data quality (Thao 2013, PASCO 2013) [30], [31]. Data is recorded and stored continuously during the survey that will be

processed automatically later on in the office except pavement crack ratio to formulate pavement condition dataset for all sections of 100 lane-meters long.

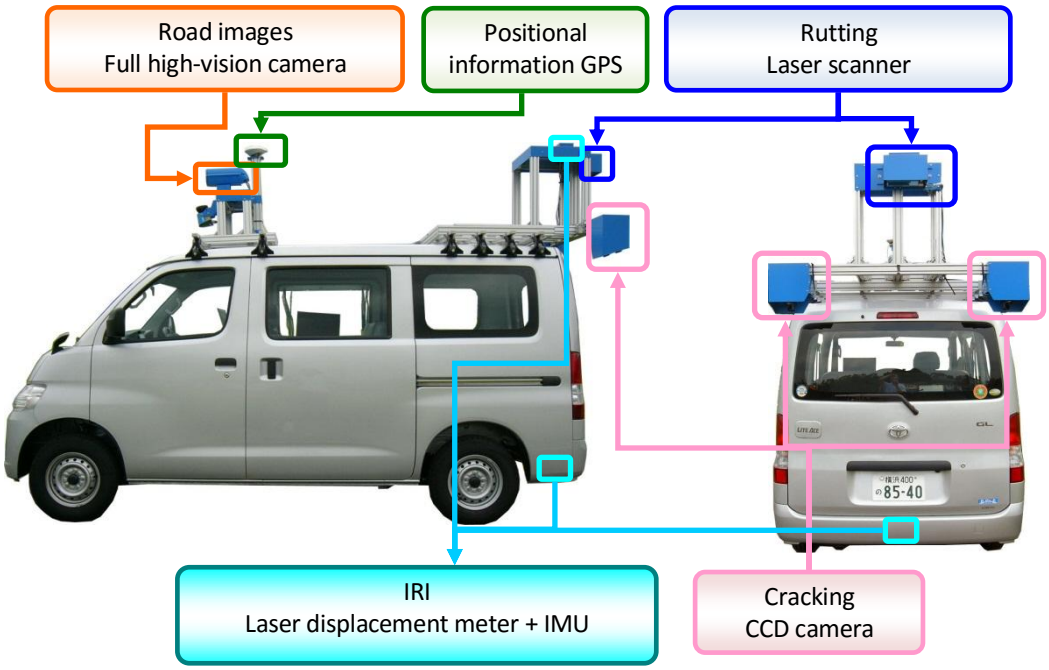


Source: PASCO, 2013

Figure 5.43 Flowchart of Pavement Condition Survey

Beside three pavement indicators of IRI, rutting, cracking, some other data are also collected during the survey such as road coordinators and road front images with capture interval of five meters, locations of road facilities, and some inventory data (Figure 5.44). The overall pavement indices can be calculated from the three individual indicators to evaluate pavement soundness and the need of repair works. Collaboration scheme as mentioned in Figure 5.15

had been applied for this activity also for fully technology transferring to local engineers who will take the whole responsibility for deployment the technology after the project.



Source: PASCO, 2013

Figure 5.44 Overview Image of the Pavement Survey Vehicle

5.5 PROJECT OUTCOMES

It is quite clear that by incorporating all five components, the practical project covers and challenges a comprehensive scope of road management and maintenance in Vietnam that consists of both technical contents and institutional issues with the objectives of enhancement road maintenance capacity in the targeted region and dissemination capacity to expand project outputs across the country. Within the tight schedule of the project, all expectation has been satisfied as the results of great efforts from both sides and valuable academic contribution (Figure 5.45).

Regarding to the first component of road database system, In the past, central road administrator had decided to use RoSyBASE as their road database system. However, because of various technical problems, it has no longer been operative. Database is the heart of any management system; it can be best utilized for various purposes such as asset management and traffic operation management. With this, the project developed road asset database, focusing on the development of database formats and data input software system. The structure of road asset database comprises four main components of data: general road management, road asset

or road inventory, pavement condition, maintenance history and traffic volume data. (Figure 5.46) A large amount of available data has been computerized into the new road database system.

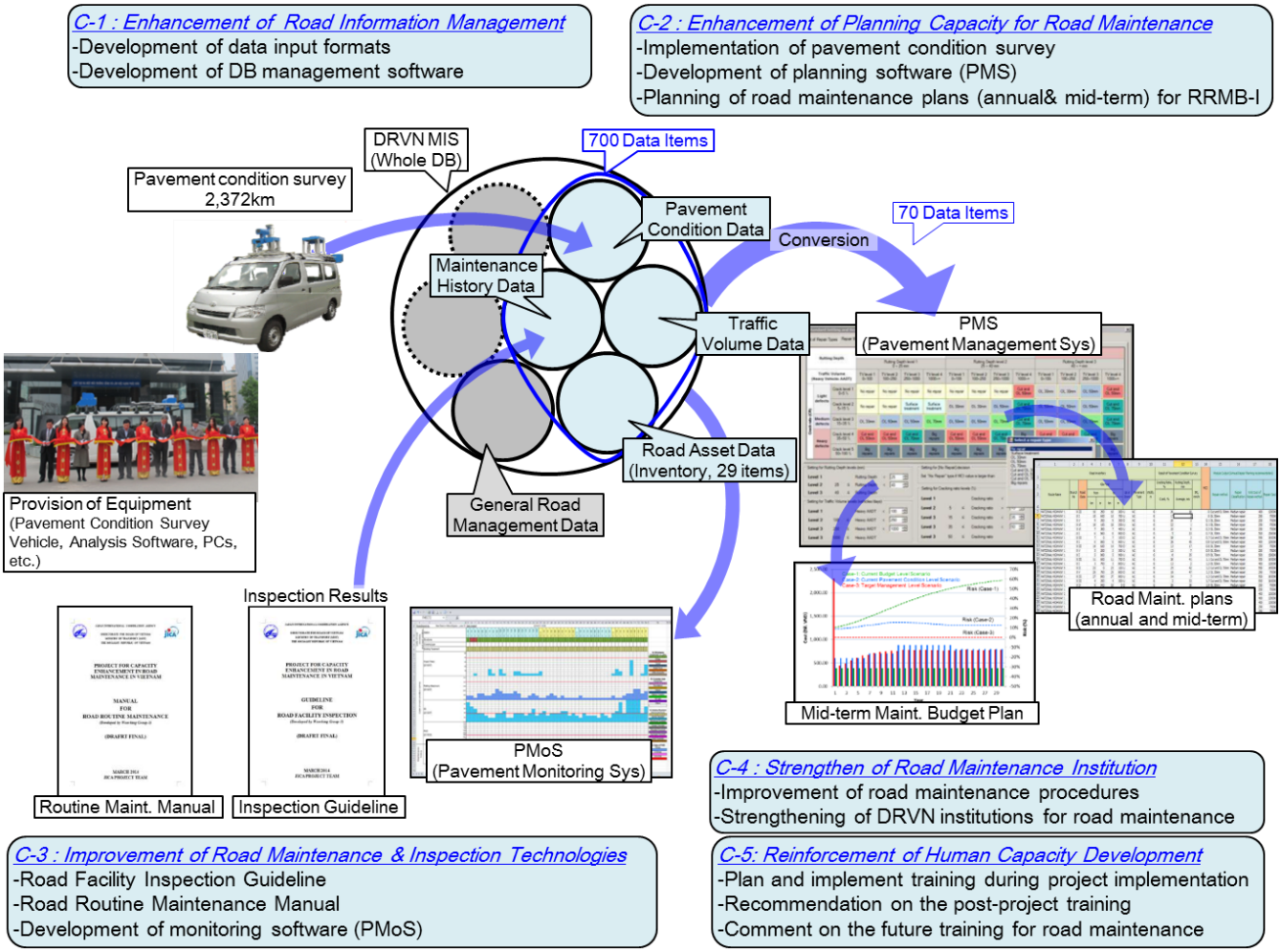


Figure 5.45 Project Components and Outcomes

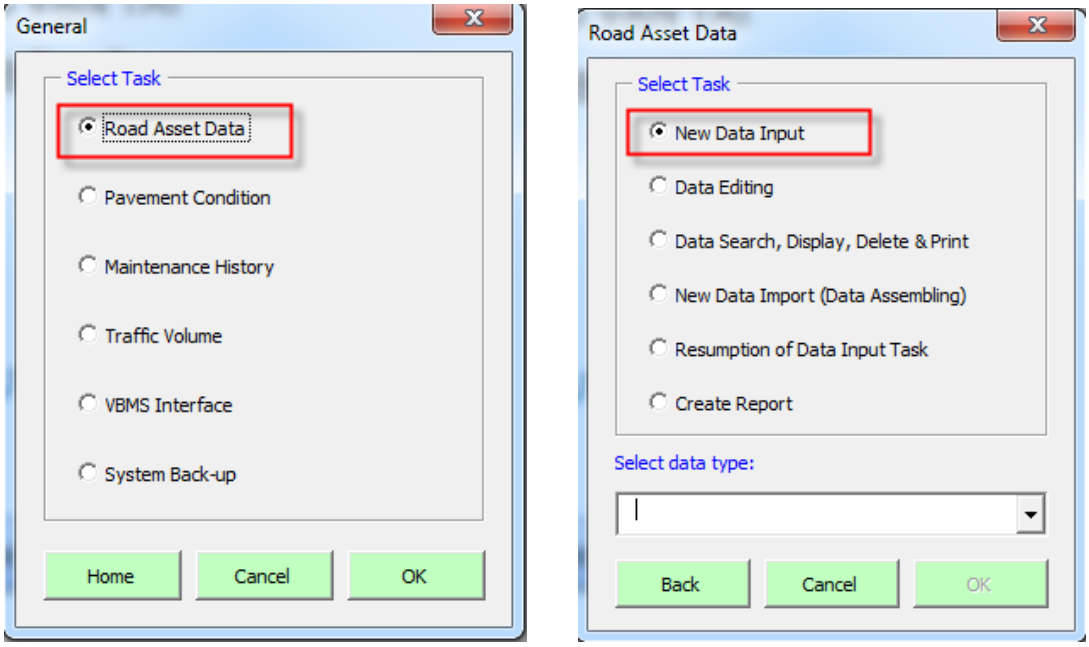


Figure 5.46 Interfaces for New Input of Road Asset Data

With the focus on road maintenance planning enhancement, all outputs from project's second component have been successfully conducted or developed that consists of pavement condition survey applying automatic technology, data conversion software and new PMS system (Figure 5.47, Figure 5.48, Figure 5.49, Figure 5.50).

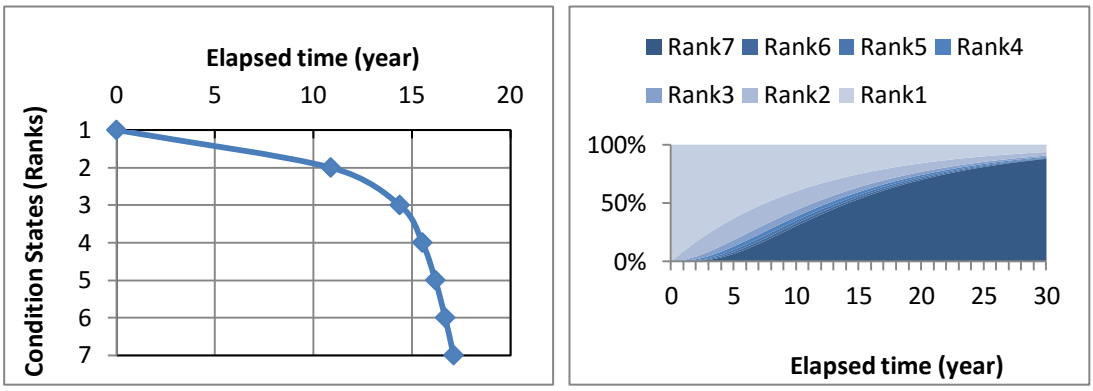


Figure 5.47 Predicted Performance Curve and Transition of Pavement Condition in IRI

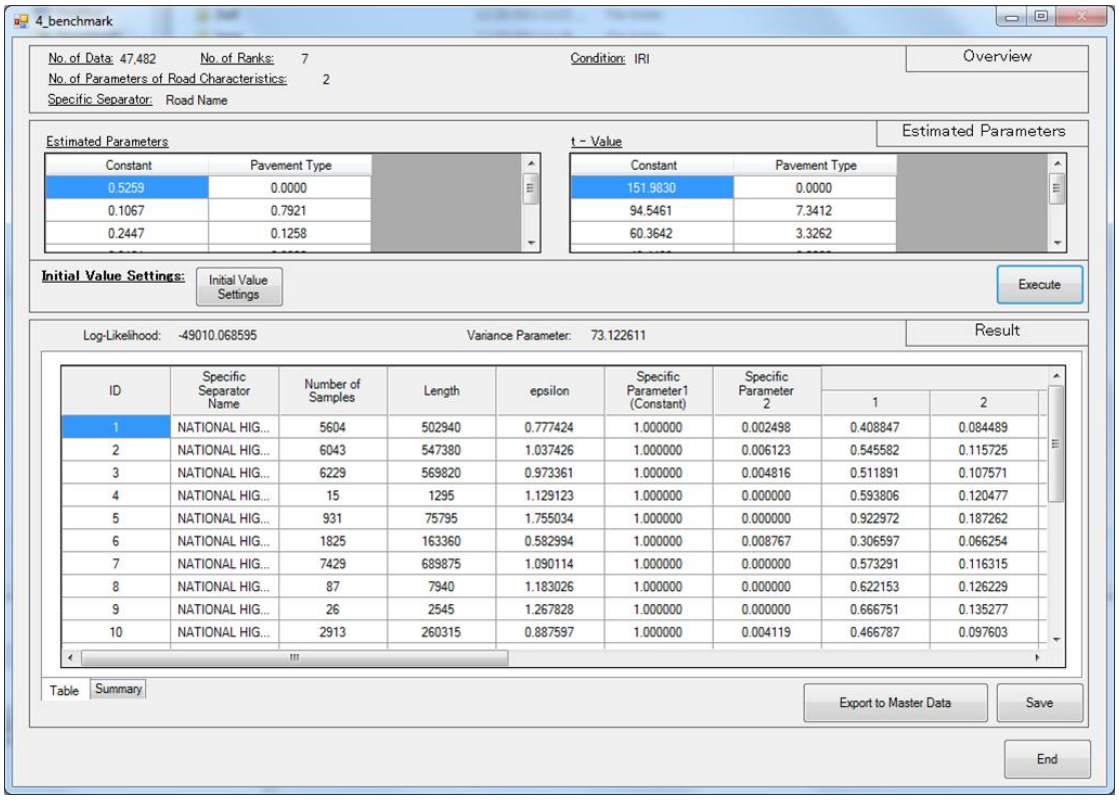


Figure 5.48 PMS Interface to show Benchmarking Execution for IRI

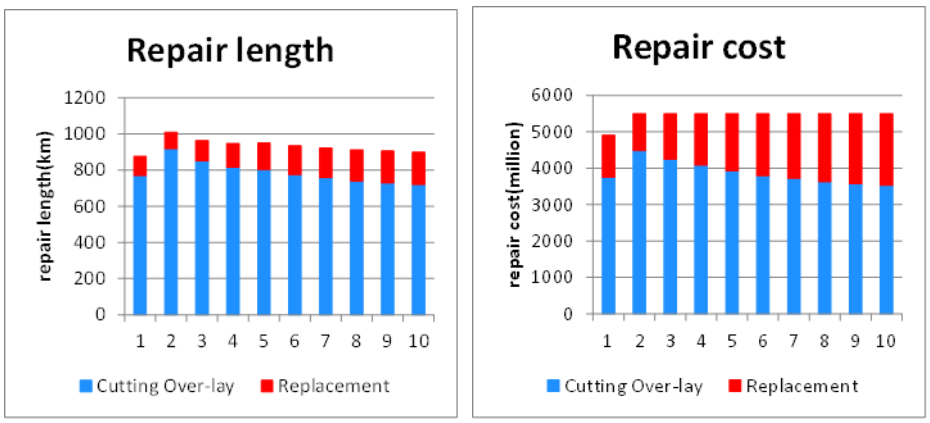
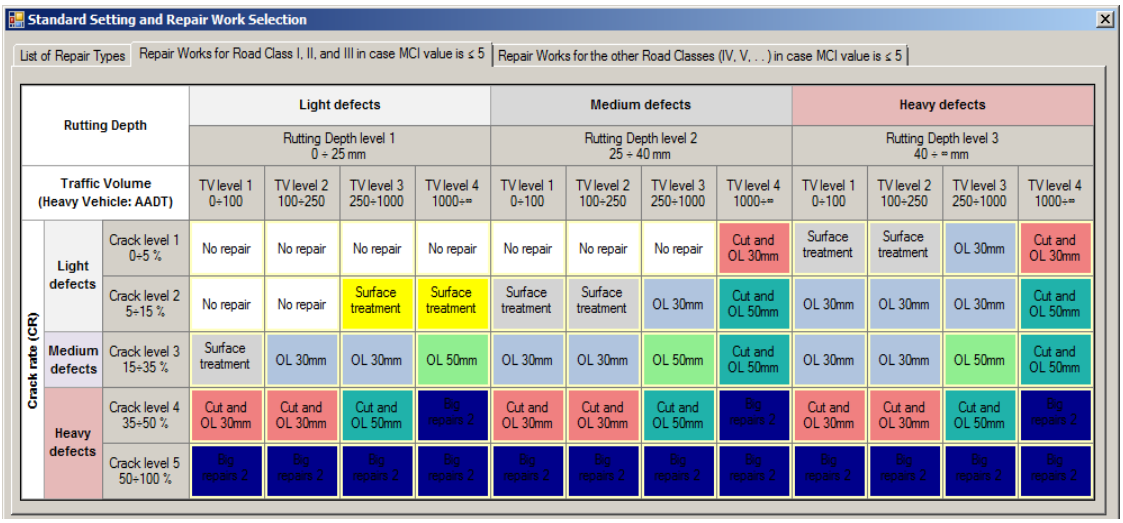


Figure 5.49 Results of Budget Simulation: repair work and repair cost



Standard Setting and Repair Work Selection

List of Repair Types | Repair Works for Road Class I, II, and III in case MCI value is ≤ 5 | Repair Works for the other Road Classes (IV, V, . . .) in case MCI value is ≤ 5

Rutting Depth			Light defects				Medium defects				Heavy defects			
			Rutting Depth level 1 0 ÷ 25 mm				Rutting Depth level 2 25 ÷ 40 mm				Rutting Depth level 3 40 ÷ ∞ mm			
Traffic Volume (Heavy Vehicle: AADT)			TV level 1 0÷100	TV level 2 100÷250	TV level 3 250÷1000	TV level 4 1000÷∞	TV level 1 0÷100	TV level 2 100÷250	TV level 3 250÷1000	TV level 4 1000÷∞	TV level 1 0÷100	TV level 2 100÷250	TV level 3 250÷1000	TV level 4 1000÷∞
Crack rate (CR)	Light defects	Crack level 1 0÷5 %	No repair	No repair	No repair	No repair	No repair	No repair	No repair	Cut and OL 30mm	Surface treatment	Surface treatment	OL 30mm	Cut and OL 30mm
		Crack level 2 5÷15 %	No repair	No repair	Surface treatment	Surface treatment	Surface treatment	Surface treatment	OL 30mm	Cut and OL 50mm	OL 30mm	OL 30mm	OL 30mm	Cut and OL 50mm
	Medium defects	Crack level 3 15÷35 %	Surface treatment	OL 30mm	OL 30mm	OL 50mm	OL 30mm	OL 30mm	OL 50mm	Cut and OL 50mm	OL 30mm	OL 30mm	OL 50mm	Cut and OL 50mm
	Heavy defects	Crack level 4 35÷50 %	Cut and OL 30mm	Cut and OL 30mm	Cut and OL 50mm	Big repairs 2	Cut and OL 30mm	Cut and OL 30mm	Cut and OL 50mm	Big repairs 2	Cut and OL 30mm	Cut and OL 30mm	Cut and OL 50mm	Big repairs 2
		Crack level 5 50÷100 %	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2	Big repairs 2

Figure 5.50 Interface to show and customize Flowchart of Work Repair Selection

Regarding to improvement of road maintenance and inspection technologies in the third component, current situation of relevant practices in both Vietnam and Japan have been carefully studied and reviewed to develop two useful and practical documents of (i) road facility inspection guideline, and (ii) road routine maintenance manual. One practical system, road pavement monitoring system (PMoS), is another output in the third component to display or visualize data of pavement performance.

Within the framework of fourth and fifth components, there is no system or technical output as the outcome but strong recommendations on institutional enhancement and human resource development including many provided trainings have been recognized.

PMoS is actually a visualization system of road conditions by maximizing utilization of available pavement condition data, maintenance history which are visualized as bar charts along road stretched plan (Figure 5.51). One of its main objective is to strongly support maintenance practitioners to make decision at project level. Hence, management criteria or requested LOS in terms of each pavement indicators or index is integrated in the charts which demarcate road sections into two regions: (i) sufficient LOS and (ii) critical LOS need being treated. Users are not only in capable of monitoring pavement conditions by keeping data updating but also confident to propose the list of candidate repair sections for next short-term or fiscal year.

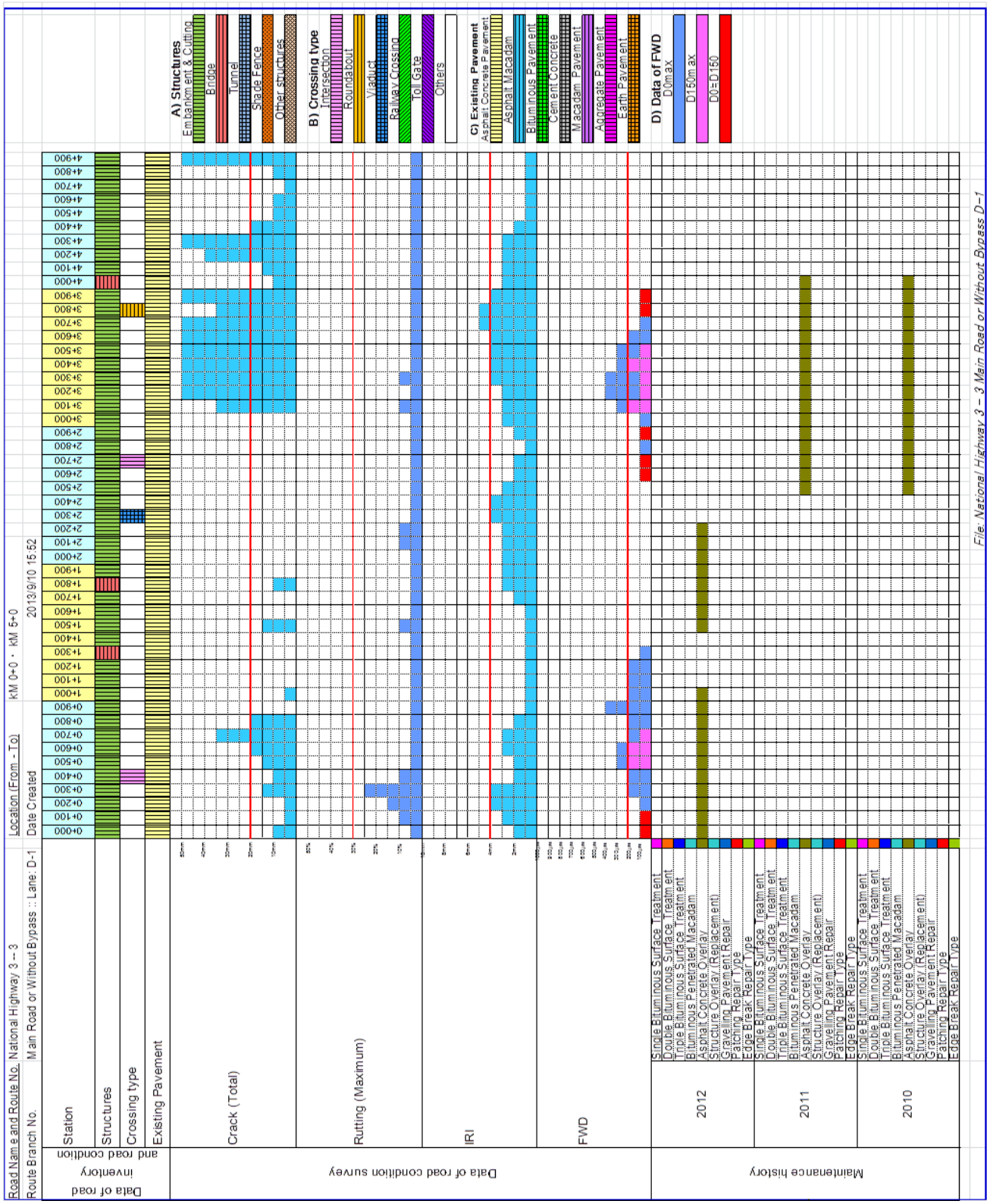


Figure 5.51 Output Display of PMoS

5.6 SUMMARY AND RECOMMENDATIONS

5.6.1 Proper Technology Transferring and Receiving for Sustainable Development

Previously, in the field of road maintenance and management in Vietnam, there had been many technical assistant (TA) projects from donors. It can be understood that TA projects are very different from cooperation projects (PC) especially in term of achievement. For TA project, both the donor and recipient mostly pay attention only to the outputs that belong to consultant's assignment and responsibility. Whereas, for PC project, outputs are important however transferring knowledge and technology is much more significant. It also means that in PC project, recipient's leading involvement and participation is the crucial point to make them understand not only the outputs but also how to realize these outputs or the full process to achieve the goals.

Moreover, in some cases even consultants took efforts for technology transferring but it was just one way because of lacking proper receiving motivation and capacity of receivers that also could not secure for any sustainability. After many TA projects, everything had gone.

From lessons learned in previous other donors' TA projects, it was decided to the formulate PC project and took proper arrangement for project human resource that is formed from collaboration between Japanese experts and counterpart's selected members. During project implementation, all relevant issues have been raised for discussion to come to common agreement. On the job trainings have been intensively implemented through the whole project duration to make sure for sufficiently technology transferring and also to improve R/D capacity for local organizations including formulation trained trainers.

5.6.2 Enhancement of Human Capacity by Continuous Training Program

Moreover, collaboration with many non-recipient organizations has been strongly emphasized especially to introduce the new PMS in Vietnam. It also has been strongly confirmed that out of the project framework, jointly annual summer courses on road infrastructure asset management since 2005 so far for Vietnamese engineers, practitioners and researchers on voluntary based makes great contribution not only for enhancement of human capacity but also for increasing awareness of decision makers and public on road asset management and maintenance. During project implementation, to facilitate best for realizing project goals, direction of contents for the training courses have been customized also with more focus on Kyoto model that covers comprehensive issues from fundamental knowledge and theory, models building, practices and trial running to make many learners have good chances not only to understand the system but also to get motivation and involvement. Such scheme is expected be disseminated and applied in many other places.

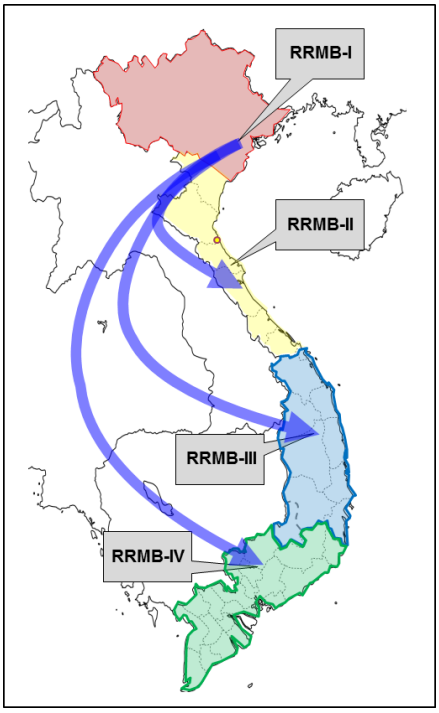


Figure 5.52 Dissemination Project Outcomes from Pilot Region to Nationwide

Within the framework of the practical project, national road network in the northern region under RRMB-I had been selected as the pilot region. It is planned that many trained trainees in the project will play the key role to disseminate their obtained technology to other regions by the own local efforts (Figure 5.52).

5.6.3 Customization to Fit Local Conditions

There are various practices on implementation of Kyoto model in Japan before the selection for development one new PMS system in Vietnam based on its platform.

In theoretical approach, the methodology is the unique for all applications with the core of deterioration forecasting model. However, in real practice, contents and components of PMS can be different from each system depending on the conditions and constraints of local application.

Firstly, the system should be operable, well functional but simple without unnecessary burden to road administrators to avoid losing their initial motivation especially on data preparation that has been realized as one of the main cause for failure in application of previous PMS systems in Vietnam. Available information and data in hardcopy have been taken its full advantage to formulate one time-series dataset instead of expending great resources of time, budget and human force for the second pavement condition survey. Moreover, some minor function such as illustrative display on WEB-GIS had been decided not to propose for releasing data preparation work at the initial stage.

Secondly, utilization of benchmarking analysis has been maximized in factor analyzing for intensive and comprehensive evaluation of pavement deterioration to customize local requirement on mid-term planning of proposing not only budget plans but also the detail list of repair volume or sections. Such demand on outputs of mid-term or strategic planning is quite similar to tactical implementation of annual planning as the constraint of regulations in the State Budget Law.

One more main point of customization is the setting of criteria to evaluate pavement condition and flowchart of work repair selection (Figure 5.29, Figure 5.30). There is no different in the approach but these pre-set values must reflect not only technical issues such as current practices of pavement maintenance, design specifications but also national socio-economic status.

In the reality of implementation road management in the future, there will be raising requirements on the new developed system that demands for its continuous customization. It seems that such further demands will mostly focus on: (i) further expanding its function such as WEB-GIS integrated, data processing and calculation issues, (ii) customization for application to other road systems such as expressway, urban roads and local roads, and to other road facilities.

Moreover, in the rather long future, with the more involvement of private sector in transport infrastructure investment, management and operation, PMS system can be requested to customize taking into consideration of road authorities, owners or maintenance contractors.

There is no doubt about blooming perspective that other countries specially in Asian region will also do such similar approach by initiating from the basic platform of the developed PMS in Vietnam. In such case, the customization demand is more specialized.

For the vision of sustainable implementation of the PMS system, two main issues had been taken into account during PMS development to satisfy various kinds of demands for customization, improvement as well. Open-source based is the highest priority that is followed by organizing PMS system in different modules.

5.7 CONCLUSIONS

Long experience with big lessons learned regarding application of closed asset management system leads to definitive decision to approach to open system as the new direction. From beginning state of development PMS system, new one based on platform of Kyoto model now is available for operation as the achievement of fruitful cooperation between Japan and Vietnam to enhance road management and maintenance in Vietnam.

The new PMS system is also consistent with the new international standard for asset management ISO55000[12] that was published in January 2014. The system adopts methodology that estimates deterioration model by using repair history data, and available datasets of pavement conditions periodically collected through its life cycle. The estimated performance curves explain the tendency that pavement deteriorates. Budget planning for different maintenance scenarios will be made based on performance curves and repair policy or flowchart of work repair selection taking into account of priority of candidate repair sections. Benchmarking evaluation is also useful function to identify target groups of pavement sections with fast deterioration speed to formulate plan of detail investigation as well as to verify appropriateness of pavement technological alternatives under certain conditions.

It is highly emphasized that based upon technology transferring within the practical project and continuous supports from Japanese researchers and experts, engineers in Vietnam road authorities can easily touch the system to customize it by changing pre-set values such as budget constraint, pavement ranking definition, repair policy and selection, and so forth especially upgrade and expansion the system to fit new demands in the future.

It is time for road authorities in Vietnam to take their turn to disseminate outputs of the project including new PMS system to put into operation. Training provision by central authority to regional ones should be considered as the key issue.

Moreover, negotiation with competent organizations for officialization or institutionalization of the system should be made to promote for official application with the approval for budget plans prepared by the system and new item of budget allocation for periodical PMS dataset preparation. It must be understood that data preparation is not free or charge, there must be sufficient financial arrangement in conformity with the requirement of data for PMS system. Informative calculation results from PMS system especially work effect model to clarify the contribution or effect of each budget investment scenario to road performance should be given to the public for raising awareness about significance of road maintenance and protection as well as their tax paying.

In addition, there should be some proper institutional arrangement to set assignment of system management including upgrading and expansion to proper unit with sufficient R/D capacity. And keep doing collaboration with relevant agencies seems to be indispensable.

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CHAPTER 6

Implementation of PDCA cycle for Enhancement of Pavement Design in Vietnam

6.1 GENERAL INTRODUCTION

In order to study behaviors of pavement structure and materials, various tests have been established for simulation. However, there is no testing model that can simulate the accurate behaviors under actual traffic and environmental conditions in the field. Therefore, data of pavement conditions collected in the field that characterizes its behavior in reality must be significant and any effort on data work of collection, management, preservation, and so forth is highly appreciated. These data can be used to grasp current status of road infrastructure as the common task supporting short-term implementation of pavement maintenance, and to evaluate or estimate the deterioration progression in the future in order to prepare longer term maintenance plans or strategies. Once completion of planning, doing maintenance should be implemented that is followed by periodical monitoring or checking including data accumulation and database enrichment to support for the next planning to complete one cycle of pavement management and maintenance or PDCA (Plan – Do – Check – Act) cycle.

Implementation of PDCA cycle becomes so significant because any relevant activities can be optimized to improve pavement management and maintenance continuously that may increase pavement longevity, minimize its LCC, and strengthen accountability to road users and public. By applying post-evaluation, there is opportunity for any predecessor activity being improved by its following ones. In the PDCA cycle, post-evaluation based on accumulated data from "Check" would bring useful information and valuable recommendations to take action ("Act") for enhancement of "Do" or "Plan" as well. Over the time, PDCA cycle would be enhanced also.

As shown in Figure 6.1, at the initial stage, PDCA cycle was established that consists of many activities and components. Recognition "Plan", "Do", "See" and its tasks is quite simple because it is practical, understandable and be visual. However, in many cases, it is not so easy to deeply understand "Act" especially what kind of action, how to take action, when to take action and its effects as well as influences. In comparison with other three components of "Plan", "Do", "See", "Act" is more hidden but very significant for any improvement because it is the field of making decisions that is supported by post-evaluation in "Check" mostly. In the

first enhancement of PDCA cycle, "Do" is the main target for improvement based on some decisions such as: expanding good pavement technologies based on verification of trial application to the national wide, applying new maintenance contract model of PBC (Performance Based Contract), and so forth. The second shifting of PDCA cycle to advanced stage is made by the dramatically improvement or innovation of "Plan" that can be explained by the introduction of systematic planning models instead of subjective models.

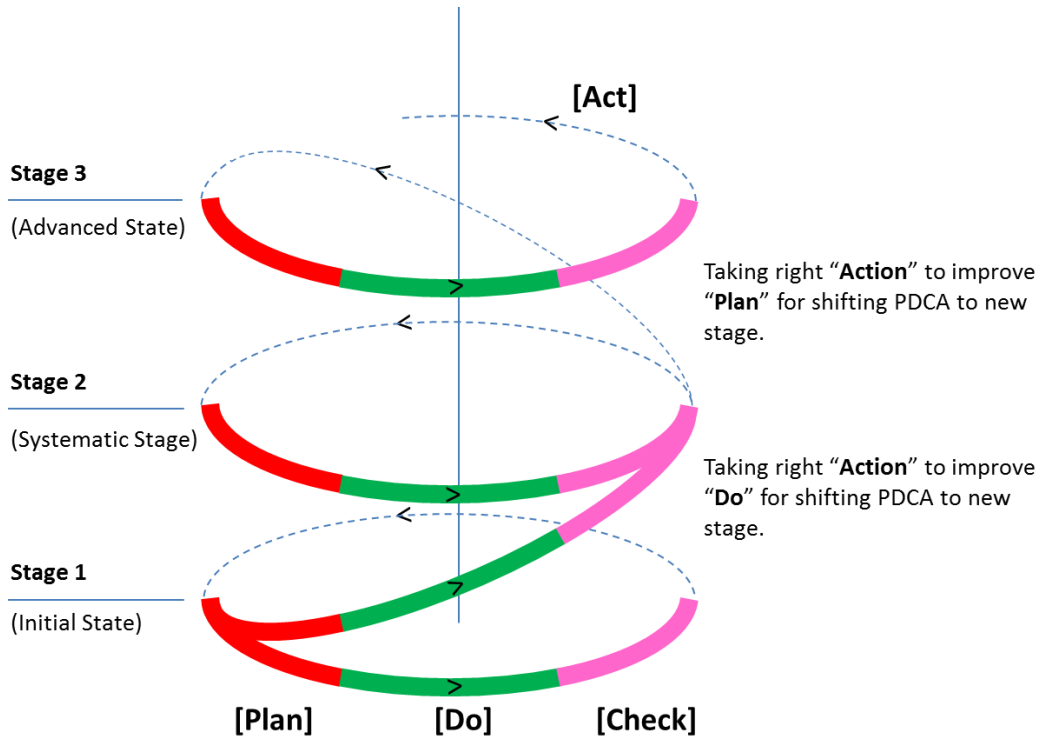


Figure 6.1 Hierarchical PDCA management cycle in improvement

Influences from improvement of PDCA cycle should be understood in a broader scope for many relevant activities because post-evaluation also creates feedbacks to any related issue. For example, by grasping the behaviors of pavement in the field in reality not only gives feedbacks to construction work but also to pavement design work for finding out any point for improvement such as: structural response model, performance model, material properties, criteria to verify trial design, mixture design, etc.,

In this chapter, the main research focuses on implementation of PDCA cycle to enhance practices of pavement structure design in Vietnam based on post-evaluation of pavement performance using rich database of surveyed pavement conditions. Hereinafter, Section 6.2 introduces briefly of pavement structure. Practices of flexible pavement designing general and in Vietnam are described in Section 6.3. Section 6.4 shows the intensively empirical study for improvement of pavement structure design including assessment about the needs or demand for improvement. The last section summarizes the key points and describes the recommendations to be considered for enhancement of pavement design in Vietnam.

6.2 INTRODUCTION OF PAVEMENT STRUCTURE

Typical pavement usually consists of surface course, and foundation built over compacted subgrade as shown in Figure 6.2 (Michael S.Mamlouk, 2006) [1]. Surface course is widely made of hot-mix asphalt or asphalt concrete for flexible pavement, and cement concrete for rigid pavement. In the road foundation, there could be some materials of base course and sub-base course. In some cases, sub-base is not required, whereas in a quite few number of cases both base and sub-base layers in the foundation are omitted. While surface layers are bond materials, the material for the base course is mainly unbound aggregates. However, in case of necessary for improving its performance or loading capacity, the aggregate base could be stabilized with cement, bitumen, lime, or chemical agents. In order to optimize pavement structure, selection of local aggregate or in-site materials is usually encouraged for sub-base or the layer beneath this course.

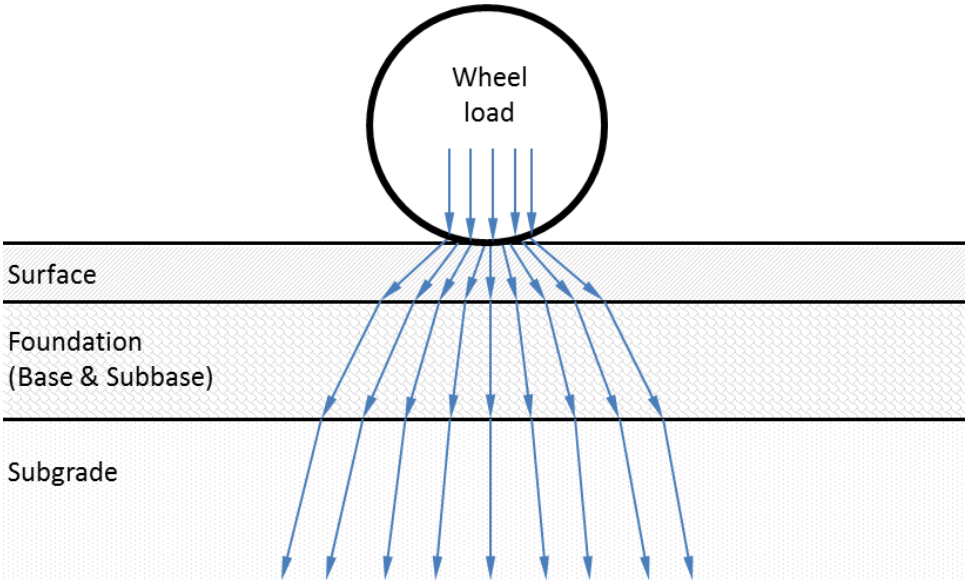


Figure 6.2 Distribution of Vehicle Wheel Load in Flexible Pavement

Both flexible and rigid pavements are designed as all-weather and long-lasting structures to serve everyday traffic during its desired service life. There are requirements on pavement riding comfort or functional performance for safe and convenient travel, and structural capacity to properly distribute vehicle wheel loads in the manner that the induced stresses transmitted to the subgrade are of acceptable magnitudes (T.F.Fwa, 2006) [2]. Due to the difference in material properties between flexible pavement and rigid one especially in term of its stiffness, the mechanisms of load transmission are very much different.

In flexible pavement, when the wheel load is applied on its surface, a localized deformation bowl occurs right around the contact area between the tyre and pavement surface, and the wheel load is distributed gradually with depth on larger area. Hence, stresses in pavement

structure caused by wheel load decrease from the top with highest value to the bottom as the depth increases. In pavement design, the principle for arrangement of material layers must be in conformity with such mechanism. The best quality materials must be located at the top and as the depth increases, lower quality materials are arranged. Bond materials are also selected for the surface due to the high wheel load induced stresses and higher requirements on abrasion and aging resistance, texture performance and so forth.

Unlike flexible pavement with the requirement on sufficient thickness to distribute the wheel load with depth, rigid slab of Portland cement concrete is the main component of rigid pavement in spreading applied load over a much more wider area on its foundation to make a significant reduction of stresses through the slab.

Both flexible and rigid pavements are designed to withstand repeated vehicle loadings. In flexible pavement, each repetition of vehicle load makes contribution to some extent to pavement distresses or defects. After each loading, a very small amount of deformation could remain permanently which could accumulate over time under load repetitions causing pavement rutting in the wheel path, and other deformation. On the other hand, the main design consideration of rigid pavement is the fatigue failure of pavement due to repeated loadings. Moreover, thermally induced tensile stresses are also another major design concern for cement concrete pavement.

6.3 PRACTICES OF FLEXIBLE PAVEMENT DESIGN

6.3.1 Classification of Pavement Design Methods

In highway engineering, beside geometric elements design, pavement design is also one significant activity due to its big and sensitive influences to many relevant issues both technical and financial. Haas et al,. (1994) [3] summarized the four main objectives of pavement design as follows:

- Maximum economy, safety, and serviceability over the desired service life
- Maximum or adequate load bearing capacity in both terms of its magnitude and repetitions
- Minimum or limited deteriorations over its desired service life
- Minimum or limited both noise and air pollution during construction and operation

In the reality of pavement design practices, there are many design methods that have been developed by various institutions. In general, it is understood that there are five distinct approaches for flexible pavement design have been introduced for application as follows:

- Experience based methods
- Methods based on soil formula or simple strength tests

- Empirical methods
- Mechanistic analysis of layered systems
- Mechanistic-empirical methods

6.3.1.1 Experience Based Methods

This is the most simple method of pavement design. Standard pavement sections for various range of traffic intensity and natural conditions have been adopted (T.F.Fwa, 2006) [2]. Based on past experience of pavement practices, standard sections are selected being applicable to local materials and budget practices. Changing of pavement serviceability with age is not considered in these methods. Due to its simplicity, low design cost with reliability at certain level, there still exists scope of application this method in small-scale projects.

6.3.1.2 Methods Based on Soil Formula or Simple Strength Tests

Empirical relations between the subgrade properties (soil classification, loading capacity) and required pavement thickness are used in these methods with the assumption that vehicle load must be born mostly by the subgrade, whereas pavement layers are significantly used to secure for its functional performance in riding comfort and dust control. Similar to the experience based methods, there is no consideration of pavement serviceability changing with age in this approach.

6.3.1.3 Empirical Methods

In this approach, extensive field observation and intensive monitoring of pavement performance under various conditions of traffic intensity in both magnitude and repetitions, natural conditions including subgrade and existing ground properties must be conducted. A large amount of obtained data is used to develop empirical relations between pavement thickness and material properties, traffic, and environmental conditions.

Of this approach, the 1993 AASHTO Pavement Design Guide (AASHTO, 1993) developed by the American Association of State Highway and Transportation Officials is one typical methodology. This empirical pavement design methodology was made based on AASHO Road Test (American Association of State Highway Officials, the former of AASHTO) with extensive scale in the late 1950's. The primary purpose of the AASHO road test was to determine the relationship between vehicle axle loading and pavement structure on its performance under the need to assist in the design of pavements, and to provide an engineering basis for establishing maximum axle load limits, as well as to provide a basis for the allocation of highway user taxation (John P. Hallin *et al*, 2007) [4]. Six test loops consist of 138 pavement sections had been constructed in Ottawa, Illinois, in the U.S. from 1958 to 1960 for empirical studies with the interval of two weeks for pavement condition surveys. In Japan, the

asphalt pavement design (T_A-method) is also another empirical method. It is found largely on the basis of domestically developed technology incorporating with principles of the AASHO Road Test (Japan Road Association, 1980) [5].

Pavement serviceability changing with age is taken into account in this approach that assists designer to design the pavement structure to serve for a certain life with a preset serviceability level. Another advantage of this approach is the possibility for economic analysis to make comparison among proposed alternatives of pavement structure.

In spite of the advantage of empirical methods in comparison with previous approaches, there still exists limitation of expanding the developed empirical relations to new application condition that is different from the applied conditions in empirical studies. If difference is recognized in any input parameters such as the change of design vehicle in axle loads, tyre pressure or if new materials are proposed, as well as the critical conditions of nature are exposed, the empirical method would not be valid or low reliability of pavement design may occur.

6.3.1.4 Mechanistic Analysis of Layered Systems

The approach of mechanistic analysis in pavement design is purely fundamental with the consideration and analysis of basic material responses of stresses, strains, and deformation under applied vehicle loads. The load is applied on a simulated layered-system to determine critical responses within the pavement structure. Material performance or capacity is used in comparison with corresponding critical responses to verify that the proposed pavement structure would serve for traffic for the desired service life without failure.

In this approach, many methods have been made so far to determine critical stresses, strains, and deformation or deflection in multi-layered system of pavement. In very early stage in 1885, Boussinesq proposed the methodology to calculate the vertical stress within a half-space or a single-layer system caused by static point load. The assumption of homogenous, isotropic, and linear elastic properties had been applied for all materials. Surface deformations can be calculated also.

Limitation in single-layer system in Boussinesq's method had been solved by Burmister. In 1943, he provided the sophisticated methodology of analytical expressions for stresses and displacement in two- or three-layer elastic systems due to static loads (T.F.Fwa, 2006) [2]. For simplification, calculation tables, graphs or nomograms have been developed by other researchers (Huang, 2003; Yoder, E.J. and Witczak, M.W, 1975) [6], [7]. Assumptions in their researches are also different (T.F.Fwa, 2006) [2]. In term of loading type, not only static load but also constant-magnitude moving, and dynamic loads had been proposed. Assumption of material model had been extended from linear elastic to non-linear with elastic, visco-elastic,

and plastic behavior. Regarding to boundary conditions, there consist of finite and infinite depth of the bottom layer, as well as friction at the interface between material layers.

Pavement design standard in Vietnam is one type belongs to mechanistic approach. It was first developed in 1990 based on reference to Russian pavement design standards. Section 6.2.2 describes in more detail about this current standard.

6.3.1.5 Mechanistic-Empirical Methods

After introduction of the empirical and mechanistic approaches for pavement design, both these two approaches have been applied widely in parallel for long time.

The rapid growth of heavy truck traffic and its repetitions in combination with critical climatic changes or weak subgrade have challenged road infrastructure and caused many problem of early and critical failure to highway pavement to make it fail long before the desired design life [8].

Such rapid changes would not be dealt by empirical methods due to its limitation of application conditions depending to the scale of emperical studies or tests. Zhang et al. (2000) [9] have found that the Equivalent Single Axle Load (ESAL), applied to quantify damage equivalency in terms of serviceability or even deflections in the 1993 AASHTO Guide, is insufficient to represent the complex failure modes of flexible pavements. Using the pavement serviceability index PSI, a non-measurable and subjective-based performance index determined by panel of raters, to evaluate pavement quality is less proper than objective pavement distress indicators.

In order to solve the limitations of empirical methods, new methods of mechanistic-empirical in pavement design have been approached. Among various efforts, AASHTO is the typical case of the new approach with the new Mechanistic-Empirical Pavement Design Guide (MEPDG) published in 2004. This new design methodology incorporates mechanistic principles and analysis including calculations of pavement stresses, strains and deformations responses (structural response model) using site-specific climatic, material, and traffic information and characteristics. These responses are used in damage models to accumulate damage over the design life. The accumulated damage at certain time contributes to specific distresses such as rutting, cracking, which would be empirically predicted using field-calibrated distress model (T.F.Fwa, 2006) [2]. MEPDG allows calibration of the distress models in order to make sure that the design method would represent each region's unique conditions of application.

6.3.2 Practices of Pavement Design in Vietnam

6.3.2.1 Introduction

Recently, in Vietnam there exist two systems of pavement design standards in parellel: (i) Vietnamese system that is compulsory application for pavement design and consists of two standards with the codes of 22 TCN 211-06 [10] and 22 TCN 223-95 [11] for flexible pavement and rigid pavement respectively, and (ii) AASHTO's Guideline for pavement design.

The first system had been formulated based on customization of Russian pavement design standards under local conditions for compulsory application in pavement design especially for all projects funded by domestic budget.

The later system of AASHTO's Guideline for pavement design had been introduced in Vietnam within framework of donors' technical assistances since Doimoi (Renovation) that have been applied to design pavements in ODA projects. To secure for the convenient application of the guidelines, MOT promulaged its Vietnamese version as flexible pavement design specification code 22 TCN 274-01 in 2001 based on re-compiling the guidelines taking some local conditions in Vietnam into account for trial application.

Since the first publishing of Vietnamese standard for flexible pavement design code 22 TCN 292-90 in 1990, it has been updated two times in 1993 (code 22 TCN 211-93) and 2006 (code 22 TCN 211-06).

Pavement design consists of three main contents: (i) pavement trial design or structure proposing; (ii) verifying pavement loading capacity based on the comparison the strengths of proposed pavement and stresses and affects caused by design load; (iii) design of material mixture. The first and the second contents are described in more detail as follows after explanation in brief of how to specify relevant parameters of traffic intensity.

6.3.2.2 Parameters of Traffic Intensity

According to the 22 TCN 211-06, flexible pavements are designed for design life that is decided based on road classes. In the design life, pavement will last without failure under estimated equivalent single axle load ESAL at a certain reliability level.

In the vehicle fleet, there consist different types of vehicles and its certain volume. Because axle load is so sensitive to pavement distresses so it is necessary to convert different magnitudes and different numbers of repetitions of all vehicles' load axles to an equivalent number of repetitions of a standard axle load through one road cross section. There are two cases of standard axle load (P), 10 tons and 12 tons. The equivalent number of repetitions of a standard axle load per day per the most critical or predominate lane (design lane) at the last

year of pavement designed life is defined as pavement designed number (N_{design}) of standard axle loads.

$$N_{design} = N_{total} \times f_{lane}^{crit} \tag{6.1}$$

where

- N_{total} is the equivalent number of repetitions of a standard axle load per day for both directions at the last year of pavement designed life or dailly two-directional standard axle load ESAL at the last year of pavement designed life.
- f_{lane}^{crit} is the critical lane distribution factor as shown in Table 6.1.

Table 6.1 Lane distribution factor (22 TCN 211-06)

No.	No. of Lanes in Both Directions	% of Standard Axle Load in the Design Lane (f_{lane}^{crit})
1.	1	100 (1)
2.	2 or 3 (without median strip)	55 (0.55)
3.	4 (with median strip)	35 (0.35)
4.	6 or more (with median strip)	30 (0.30)

Note) for intersection, $f_{lane}^{crit} = 0.5$ must be applied.

Regulation on lane distribution factor in 22 TCN 211-06 is quite similar to the guideline of AASHTO, 1993 as shown in Table 6.2.

Table 6.2 Lane distribution factor (AASHTO, 1993)

No.	No. of Lanes in Each Directions	% of 18-kip ESAL (standard axle load) in the Design Lane
1.	1	100
2.	2	80 – 100
3.	3	60 – 80
4.	4	50 – 70

N_{total} is calculated as follow

$$N_{total} = \sum_{i=1}^{k} C_1 \times C_2 \times n_i \times \left(\frac{P_i}{P}\right)^{4.4} \tag{6.2}$$

where

- k: the number of vehicle types in the vehicle fleet

- P: the standard axle load (10 tons or 12 tons)
- n_i: the number of axle load P_i in question that need being converted into the standard axle load. In the calculation, n_i is set equal to the repitition number of vehicle type i through typical road cross section per day for both directions at the last year of pavement designed life.
- C_1: the factor of axle configuration in term of axle number within distance of three meters that is determined in the equation 6.3, where m is the number of axles in i^{th} axle bundle.

$$C_1 = 1 + 1.2 \times (m - 1) \tag{6.3}$$

- C_2: the factor of axle configuration in term of the number of wheels in one wheel bundle (for single wheel: C_2=6.4, for double wheels: C_2=1.0, for four wheels: C_2=0.38).

Dailly standard axle load ESAL at the last year of pavement designed life (N_{design}) is the main parameter in pavement designing. However, cumulative standard axle load ESAL during the pavement designed life $(N_{cumulative})$ is applied for some consideration.

6.3.2.3 Pavement Trial Design

Regarding to this first content of pavement design, pavement structure is proposed by designers that consists type of surface, type of foundation, requirement for roadbed, number of material layers for surface and foundation, arrangement of layers in the structure from top to bottom, material physical and mechanical properties and thickness of each layer in the pavement structure.

There is no requirement of calcuation or sophitiscated analysis in proposing pavement structure but the basic principle to fit to features of affecting factors like traffic intensity and natural conditions must be followed. Moreover, considerable judgment, expertise of senior and experienced engineers can be expected to propose proper structure for analysis and verifying in the next steps. Illustration of one proposed pavement structure including relevant data and information is shown in Figure 6.3.

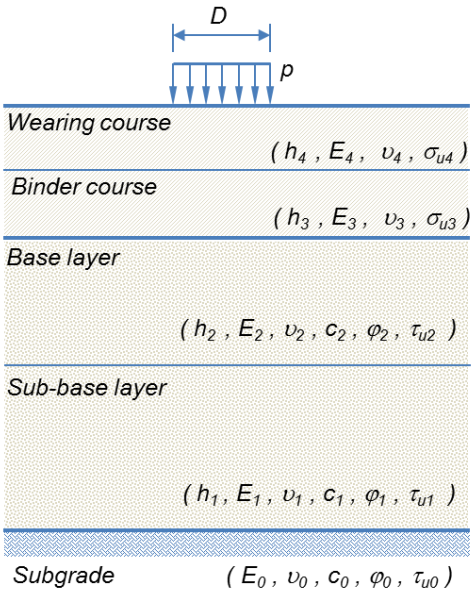


Figure 6.3 Illustration of one proposed pavement structure including relevant data and information

Pavement surface type and its minimum thickness including recommendations of its material are regulated in 22TCN 211-06 based on road class and accumulative number of ESALs as shown in Table 6.3 and Table 6.4, respectively.

Table 6.3 Selection of pavement surface type (22TCN 211-06)

Road Class (TCVN 4054)	**Pavement Surface Category**	**Materials for Pavement Surface**	**Designed Life** (year)	**Accumulative ESALs during design life** $N_{cumulative}$ (axles/lane)
I, II, III, IV	High Class A1	• Dense Asphalt Concrete (AC) type I (fine and medium size) for wearing course; • Dense AC type I or II (medium and coarse size) for binder course.	≥ 10	> 4.10^6
III, IV, V	High Class A2	• Dense AC type II, Black macadam, cold asphalt mixture with Bituminous Surface Treatment (BST) on top. • Bituminous Penetrated Macadam (BPM) • BST	8 - 10 5 - 8 4 - 7	> $2x10^6$ > $1x10^6$ > $0.1x10^6$

Road Class (TCVN 4054)	**Pavement Surface Category**	**Materials for Pavement Surface**	**Designed Life** (year)	**Accumulative ESALs during design life** $N_{cumulative}$ (axles/lane)
IV, V, VI	Low Class B1	Aggregate, water bound macadam	3 - 4	$\leq 0.1x10^6$
V, VI	Low Class B2	Compacted soil	2 - 3	$< 0.1x10^6$

Table 6.4 Minimum thickness of pavement surface – Class A1 depends on Traffic intensity (22TCN 211-06)

Accumulative ESALs for 15 years $\mathbf{N}^{15}_{cumulative}$ (axles/lane)	Min thickness of pavement surface - Class A1 (cm)
$< 0.1x10^6$	6 (5)
$\geq 0.1x10^6$	7 (5)
$\geq 0.5x10^6$	8 (5)
$\geq 1x10^6$	9 (5)
$\geq 2x10^6$	10 (5)
$\geq 4x10^6$	12.5 (7.5)
$\geq 6x10^6$	15 (10)
$\geq 9x10^6$	20 (10)

Notes): In case of there is only one AC layer in the surface, its thickness should not less than 7cm.

Thickness in () is applied for case of pavement surface directly sets on bituminous treated base with thickness no less than 10cm.

In order to control compaction quality during pavement construction, minimum thickness of individual layer must be more than 1.5 times of nominal diameter of maxium grain. Table 6.5 shows the regulation of this minimum value.

Table 6.5 Minimum and usual thickness for individual pavement layer (22TCN 211-06)

Pavement material	Min layer thickness (cm)	Usual layer thickness (cm)

AC, Black macadam	Coarse size Medium size Fine size	5 4 3	5 – 8 4 – 6 3 - 4
Bitumen chipping mixture		1.5	1.5 – 2.5
Bitumen sand mixture		1	1 – 1.5
BPM		4.5	4.5 – 6.0
BST		1	1 – 3.5
Aggregate	Dmax = 37,5mm Dmax ≤ 25mm	12 8	15 – 24
Natural aggregate		8	15 – 30
Water bound macadam		10	15 – 18
Treated soil		12	15 – 18

Even there are regulations and suggession available in 22TCN 211-06 for pavement trial design but it is not enough to propose proper pavement structure in conformity with direct issues of its loading capacity, its performance but also other considerred issues of available construction and material production technologies, available construction materials near the site, perspectives of maintenance and management, convenience for imrovement or upgrading and so forth. Therefore, senior engineers are expected to take such responsibility including reference to past practices, experience or general pavement design catalogs.

6.3.2.4 Evaluation and Verifying the Trial Design

Once pavement structure has been proposed taking into consideration of site conditions (traffic, subgrade, climate, etc,.), construction and maintenance conditions, analysis can be implemented to calculate its strengths, loading capacity and stresses, affects like rebound deflection caused by design loads of vehicle axles to verify or evaluate the satisfactoriness of the proposed pavement or how its performance meets the criteria. Pavement performance model is applied to determine capacity of proposed structure. From design load, pavement response model is analyzed to identify critical stresses or strains within pavement structure. Based on the typical distresses and defects of flexible pavements, all three following critical stages in terms of pavement capacity must be satisfied to confirm that the proposed pavement is capable enough against affecting factors of traffic intensity and natural conditions.

The first critical state: maximum rebound deformation of pavement structure under vehicle loading d must be less than the critical rebound deformation d_{crit} that is pre-set or allowed by road administrators.

$$\frac{d_{crit}}{d} \geq K_{def} \tag{6.4}$$

Pavement rebound deformation characterizes for its vertical deformation resistance under vehicle wheel loads that has inverse relationship with pavement stiffness. Therefore, the critical state in the Equation (6.4) in term of rebound deformation can be re-wrote as following equation in term of pavement modulus of elasticity.

$$\frac{E_{ove}}{E_{req}} \geq K_{def} \tag{6.5}$$

where

- E_{ove} : overall elastic modulus of the whole pavement structure that can be calculated based on its physical-mechanical characteristics and thickness of each layers.
- E_{req} : required elastic modulus for pavement that is determined based on road class, the number of estimated Equivalent Single Axle Loads (ESALs) at the end of pavement design life.
- K_{def}: cofficient of strength reserving on vertical deformation resistance.

The second critical state: maximum shearing stress in the subgrade and unbound materials caused by vehicle design wheel loads and seft-weight of on-top layers must be less than material shearing strength to secure for its elastic behavior to avoid any plastic deformation within the pavement structure and subgrade.

$$\frac{\tau_u}{\tau} \geq K_{sh} \tag{6.6}$$

where

- τ_u: ultimate shearing strengh of unbound materials.
- τ : maximum shearing stress in the subgrade and unbound materials caused by vehicle design wheel loads and seft-weight of on-top layers
- K_{sh}: cofficient of shearing strength reserving.

The third critical state: maximum tensile bending stress at the layer bottom of cementitious materials such as asphalt concrete caused by vehicle design wheel loads must be less than material flexural strengh to resist failure of structural cracking due to bending.

$$\frac{\sigma_u}{\sigma} \geq K_{tf} \tag{6.7}$$

where

- σ_u : ultimate flexural strengh of cementitious materials that can be determined by bending test in the laboratory.
- σ : maximum tensile bending stress at the layer bottom of cementitious materials due to vehicle wheel loads.
- K_{sh}: cofficient of flexural strength reserving.

Cofficients of strength reserving can be determined depending on design reliability in the following table.

Table 6.6 Determination of cofficients of pavement strength reserving

Pavement design reliability level	**0.98**	**0.95**	**0.90**	**0.85**	**0.80**
K_{def}	1.29	1.17	1.10	1.06	1.02
K_{sh}	1.10	1.00	0.94	0.90	0.87
K_{tf}	1.10	1.00	0.94	0.90	0.87

The reliability in pavement design is the chance that pavement will last for the design life or period with no failure (F.T.Fwa, 2006) [2]. In principle, the more significant road in term of its function and the longer designed life, the higher reliability level will be selected in pavement design that secure for better performance but also requires for better pavement or thicker structure in higher cost. Table 6.7 shows the regulation of minimum value of reliability level in a wide range for proper selection that fit to the conditions of application.

Table 6.7 Selection of Reliability Level for Pavement Design (22TCN 211-06)

No.	**Road Categories**	Pavement design reliability level		
1.	Expressway	0.90	0.95	0.98
2.	Highway - Class I, II	0.90	0.95	0.98
3.	Highway - Class III, IV	0.85	0.90	0.95
4.	Road - Class V, VI	0.80	0.85	0.90
5.	Urban Road - Freeway and Urban Principal Arterials	0.90	0.95	0.98

6.	Urban Road – Others	0.85	0.90	0.95
7.	Specialized Road	0.80	0.85	0.90

In case of three critical sections along the road: high supervision rate, toll plaza, stopping and parking areas, pavement design reliability must be at least one level higher than normal sections on the road.

Following Vietnamese standard for flexible pavement design code 22TCN 211-06, stresses in the pavement structure and pavement rebound deformation caused by vehicle design wheel loads or its self-weight are calculated applying theory of eleasticity for the model of the elastic multi-layer system on half-space of subgrade with the assumption of static loading, continuity conditions at the interfaces between layers, homogeneous, isotropic, and linear elastic materials. With these assumptions, only material properties are required for calculative analysis: Modulus of Elasticity *(E)*, Poisson's ratio *(υ)*, ultimate shearing strengh (τ_u), and ultimate flexural strengh (σ_u) that must be determined by testing under the most critical coressponding conditions in terms of water content or temperature. For simplification of pavement calculation, ready-made nomograms as the results of sophisticated machanical analysis for template models had been made. Pavement designers are requested to convert the real model of proposed pavement structure to the template model to apply the nomograms for determination of stresses and its elastic modulus.

Figure 6.4 shows the nomogram to determine overall modulus of elasticity or vertical deformation resistance of template pavement model that consists of one material layer with thickness of H centimeters and modulus of E_1 (Mpa) on the subgrade or elastic half-space with modulus of E_0 under loading of standard vehicle axle over equivalent circular contact area with diameter of D centimeters. Make simple calculation to enter the vertical abscissa with the ratio E_0/E_1, enter the horizontal abscissa with the ratio H/D then make horizontal projection and vertical projection respectively to identify the point of intersection to read the value of design curve that shows the ratio E_{ove}/E_1. Finally, the overall modulus of elasticity E_{ove} can be determined by simple calculation.

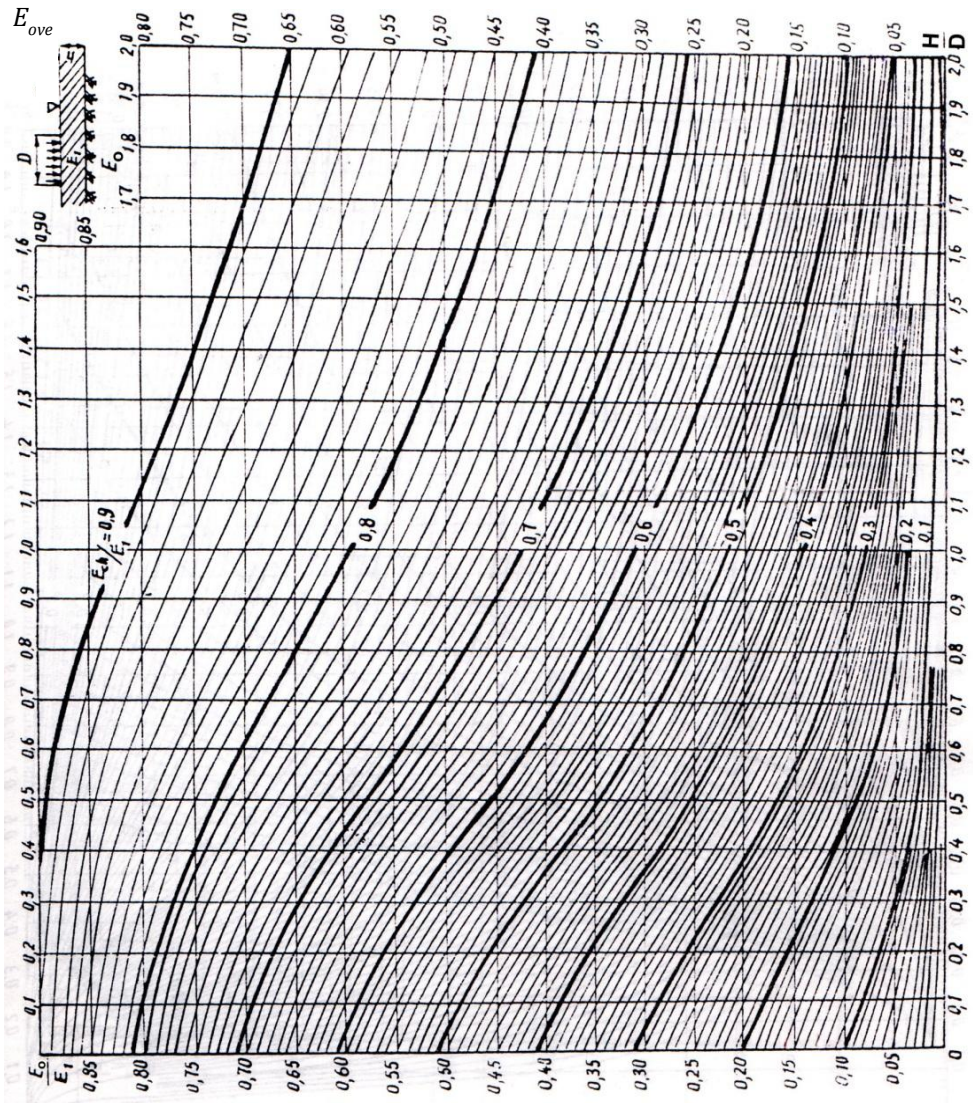


Figure 6.4 Nomogram to determine overall modulus of elasticity of template pavement model

To apply the nomogram for proposed pavement structure, designers have to convert the proposed structure to the template pavement model. Conversion is made for multi-layer structure by gradual conversion single couple of two adjacent layers to one new layer with the same total thickness but average elastic modulus E_{ave} (Figure 6.5).

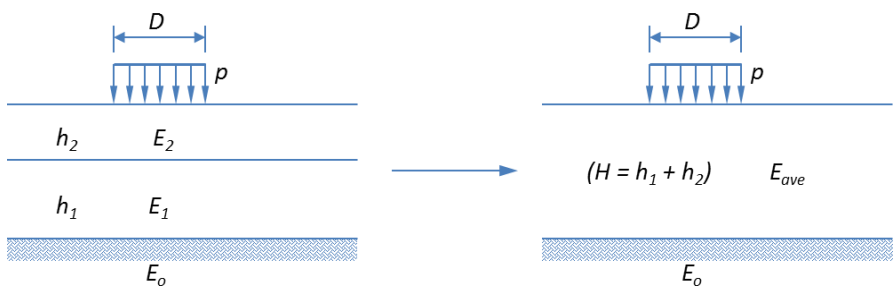


Figure 6.5 Schema of conversion two layers to one equivalent layer

The principle for conversion is stiffness or the product of elastic modulus and moment of inertia equivalency that is used to calculate average elastic modulus as Equation 6.8.

$$E_{ave} = E_1 \left[\frac{1 + \frac{h_2}{h_1}\left(\frac{E_2}{E_1}\right)^{1/3}}{1 + \frac{h_2}{h_1}} \right]^3 \tag{6.8}$$

In order to confirm structural capacity in term of pavement vertical deformation resistance under vehicle wheel loads, the critical state in the Equation (6.5) must be examined. Table 6.8 shows the matrix to determine required elastic modulus for pavement by simple interpolation with the inputs of road class, designed axle load, the number of estimated Equivalent Single Axle Loads (ESALs) at the end of pavement designed life.

Table 6.8 Matrix of pavement required elastic modulus determination (22TCN 211-06)

Standard Axle Load	Pavement Surface Category	Required Elastic Modulus for pavement in conformity with ESALs *(number of standard ESALs/day/lane at the final year of design life)*									
		10	**20**	**50**	**100**	**200**	**500**	**1000**	**2000**	**5000**	**7000**
10 tons	High Class A1			133	147	160	178	192	207	224	235
	High Class A2		91	110	122	135	153				
	Low Class B1		64	82	94						
12 tons	High Class A1		127	146	161	173	190	204	218	235	253
	High Class A2	90	103	120	133	146	163				
	Low Class B1		79	98	111						

Note) Among three types of pavement surface, B1 is un-paved with surface layer of macadam, aggregate or strengthed soil, A1 and A2 are paved surface. In which, A1 is the best surface of hot rolled asphalt concrete and in-place cement concrete, A2 mainly consists of surface layers of bituminous surface treatment or dressing, bituminous penetrated macadam or cold mix.

Regarding to determination of required elastic modulus, there exists of lacking input data of the number of ESALs in many cases due to the unavailability of traffic survey data for prediction, low reliability of prediction for long designed life or the nonsensial calculation in the case of light vehicle dominated road like urban road with nearly zero equivalent axle load factor (Equation 6.2). In this case, minimum value of required elastic modulus as shown in Table 6.9 becomes so meaning to examine capacity of proposed structure with the overall modulus of elasticity E_{ove} that is obtained by projection using nomogram in Figure 6.4.

Table 6.9 Minimum value of pavement required elastic modulus determination (22TCN 211-06)

No.	Road types and Road Classes	Pavement Surface Categories		
		High Class A1	High Class A2	Low Class B1
1.	**Non-urban Road**			
-	Expressway and Highway Class I	180		
-	Highway Class II	160		
-	Highway Class III	140	120	
-	Highway Class IV	130	100	75
-	Highway Class V		80	
-	Highway Class VI			
2.	**Urban Road**			
-	Freeway and Urban Principal Arterials	190		
-	Zone Arterials	155	130	
-	Street	120	95	70
-	Industrial and Storage Road	155	130	100
-	Cycling Road	100	75	50

6.3.2.5 Flowchart of Flexible Pavement Design Standard

The whole procedure of pavement design following Vietnamese standard is shown in Figure 6.6. Such procedure is very much different from empirical methods (The AASHTO Pavement Design Guide 1993 in American and T_A method in Japan) but it is rather similar to the AASHTO Mechanistic-Empirical Pavement Design Guide (2004) because of its mechanistic methodology. After calculation and analysis if pavement performance meets all criteria, proposed structure is confirmed to be one option of design pavement structure. The designer can repeat the procedure with other proposed structures that can suppport for making better selection of final structure by comparison taking relevant consideration into account.

It is understood that for each different condition of vehicle fleet, vehicle axle loads, number of ESALs, and natural conditions such as temperature, water content, geological features, etc., designed pavements should differ each other more or less to fit the conditions. Most these factors are requested to considered in the current Vietnamese standard. Howerver, in the standard, there is lacking of detail consideration of different traffic conditions on various pavement section types along the road such as bridge approach section, intersection approach section, bridge deck section and so forth.

In the real practices of pavement design in Vietnam for years, there exists the fact that unique pavement structure especially surface course is applied for many long routes that consist of many typical sections of different relevant factors to pavement performance. In many national

roads and along each route, typical surface course of asphalt concrete that consists of one wearing fine layer of five centimeters on one binder coarse layer of seven centimeters in thickness have been designed and applied widely for years.

Actual pavement conditions show that after putting pavement into operation, its performance and deterioration are so much different among routes and among typical sections along each route. Some kinds of typical sections such as bridge approach section, intersection approach section, toll approach section are much deteriorated with faster deterioration speed in comparison with other normal sections that leads to reduction of pavement level of service or its life longevity in whole and difficulties in spot treatment and repair for many localized and isolated sections as well as increase the potential of traffic accident in deteriorated sections and around the transition area between good and bad ones.

Road infrastructure asset management is recommended to follow the “Plan - Do - Check - Action” cycle. Checking function during road operation helps road administrators to grasp its current condition that can be analyzed to find out points for enhancement in predecessor steps of designing and construction. These quantitative feedbacks must be indispensable for road administrators to set up proper action plan, to take right action for implementation of the improvement and renew their PDCA cycle for better implemenatation of road infrastructure works in the future.

In our research, Markov Deterioration Hazard Model and Local Mixture Hazard Model have been applied to evaluate pavement deterioration in Vietnam based on pavement condition datasets to make recommendations to road administrators and competent organizations on reviewing the current Vietnamese standard on flexible pavement design for revising or supplement of detail regulation, and to enhance road construction quality management especially compaction work.

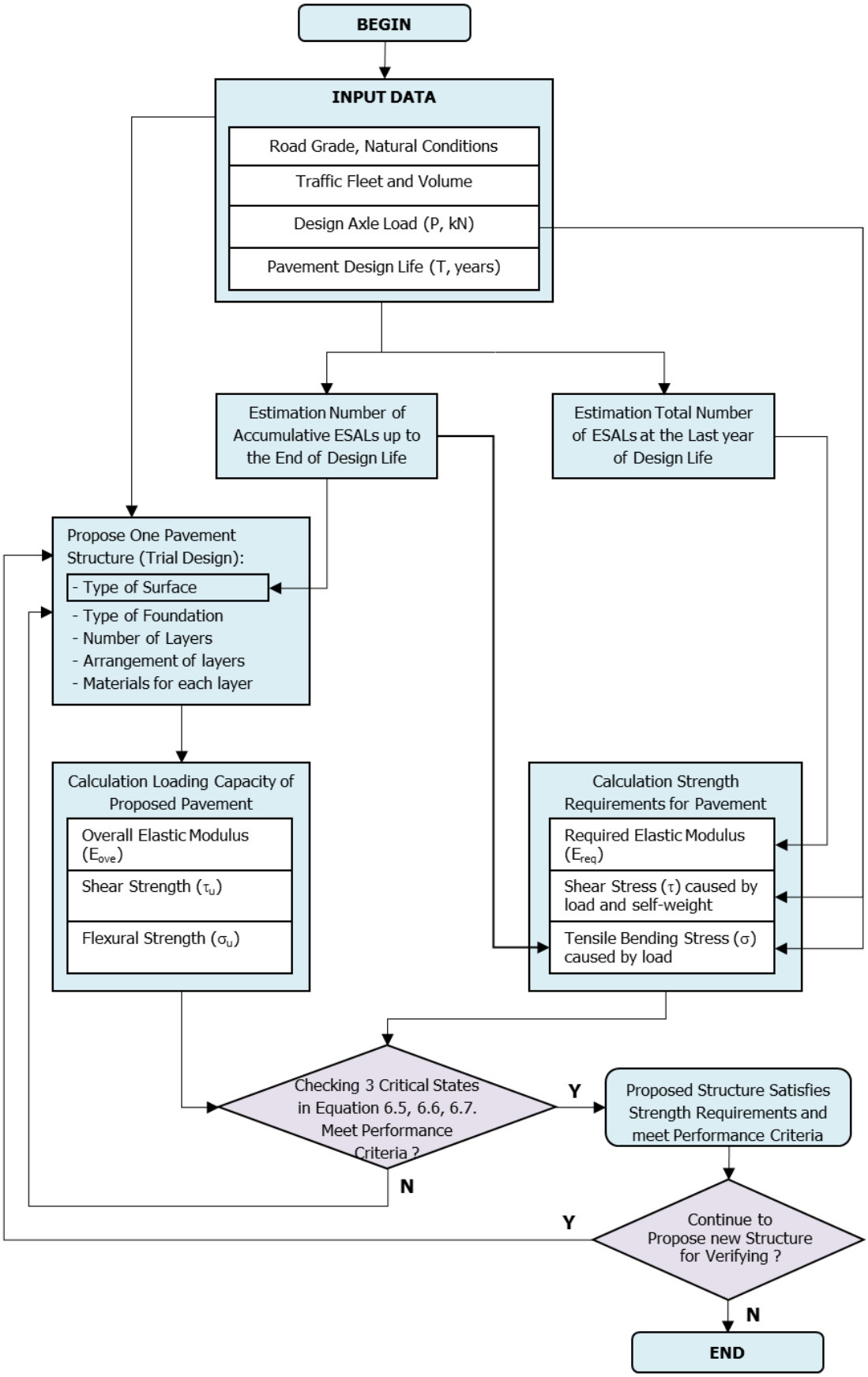


Figure 6.6 Flowchart of Vietnamese flexible pavement design standard, code 22 TCN 211-06

6.4 EMPIRICAL STUDY FOR IMPROVEMENT OF PAVEMENT DESIGN

6.4.1 Introduction

Over time, the functions of transport infrastructure facilities or assets gradually decline as damage and wear-and-tear become more pronounced. This is the so-called “obsolescence” or “deterioration” of assets. In the past, the great twentieth-century economist, Joseph Schumpeter, said that economic development was a process of creative deconstruction-a process in which new things are constantly created and replace existing ones. Innovation is the act of creating a new idea or way of thinking, manufacturing products for a lower cost, and opening up new markets. With the advancement of technology, human preferences also change. Accordingly, even if an asset had the most appropriate design for the age in which it was built, there is possibility that it will become less appropriate as times change. This is proof of social or economic development. Unlike electronic products, computers, or machinery, transport infrastructure assets have an extremely long service life. The physical functions and performance of assets deteriorate over time, but the environment surrounding these assets and influence of external factors like traffic intensity on assets change too and differ from asset by asset, location by location or section by section. A phenomenon arises in which an entire area-including assets-that was previously a “new town” ages as the humans living in the area age. As a result of this area deterioration, people’s requirements of assets change at the same time. It becomes necessary to match the content of services provided by assets to market needs to function and achieve continual improvement of asset management through the cycle of PDCA.

Along the road, there are many different types of facilities such as bridges, culverts, toll plazas, intersections, etc., that can be considerred in sectioning for homogeneous pavement sections. Road pavement faces the risk of functional obsolescence and physical deterioration. Moreover, the speed of obsolescence and deterioration varies tremendously depending on the type of pavement, location in both longitudinal and transveral directions and environmental conditions including traffic loading conditions.

Once pavement has been constructed, effort must be made to extract its maximum utility over the long period. Furthermore, the financial resources of government agencies and companies are limited, but the maintenance costs for road assets or pavements are not insignificant. There is also the risk of pavement being damaged or destroyed due to disasters. In order for government agencies and companies to sustainably continue their organizational activities under such circumstances, it is imperative that they formulate risk countermeasures to minimize costs, as well as continually evaluating the functionality and deterioration of pavement under their jurisdiction to make the feedbacks to other activities such as design work, construction work for proper solutions.

Selection for the best pavement technology or structure for each homogeneous section in the PMS is thereby, having a close link to the methodology of benchmarking analysis, which provides a managerial approach for finding out the best practice. Evidently, the core part of benchmarking study in the PMS is the application of hazard model. Because, hazard model is indispensable in estimating the deterioration of a road network, which becomes the most important key performance indicator for benchmarking.

In the network level of PMS, there are many groups or types of pavement sections, which are often categorized by the differences in enviromental conditions and traffic conditions. When applying conventional hazard models, monitoring data of the entire network is considered as a representative database used to predict the deterioration of the entire network system. This practice has some limitation that it is difficult to apply for estimating the deterioration of respective groups in the network. Especially, under the requirement of benchmarking, which aims to compare the deterioration of individual group of pavement sections in the network.

6.4.2 Pavement Conditions and Demand for Improvement

As description in Chapter 5, pavement condition survey for 23 national routes in the northern Vietnam with total length of 4,606 lane-kilometers in both directions was conducted in 2012 to grasp the current pavement performance or its deterioration, and to formulate datasets for maintenance planning.

Three pavement performance indicators of rutting depth, roughness and cracking ratio had been proposed for data collection that are also the condition for introduction of pavement index MCI.

Table 6.10 Pavement conditions - Distribution of Rutting depth

Case	Range of Rutting Depth (mm)	Number of Samples	Percentage	Accumulated Percentage	Condition
1	0≤ RUT < 10	1461	**2.97%**	**2.97%**	Very Good
2	10≤ RUT < 20	18061	**36.77%**	**39.75%**	Good
3	20≤ RUT < 30	16066	**32.71%**	**72.46%**	Fair
4	**30≤ RUT < 40**	6219	**12.66%**	**85.12%**	**Poor**
5	**40≤ RUT < 50**	3012	**6.13%**	**91.26%**	**Very Poor**
6	**50≤ RUT**	4294	**8.74%**	**100.00%**	**Serious**

Distribution of pavement rutting depth is shown in Table 6.10 with six categories of its range of value. It can be clear that rutting is very popular to be visualized by naked eyes with the depth more than ten millimeters. Critical sections in high priority for repair in big scale with rutting depth deeper than 50 millimeters are about nine percent. Candidate repair sections in rutting distress in the case 4 and 5 are around 20 percent of the total that contribute to the list of total repair sections up to 27.54 percent. This demand is really a challenging burden for the

road sector and also the state budget in Vietnam. The situation in near future would be much more critical because of a large amount of highly potential sections with the range of rutting depth from 20 to 30 millimeters. Therefore, in order to avoid huge demand of pavement repairs in the near future, preventive maintenance for rutting distress should be the priority.

Table 6.11 shows another image of pavement condition in deformation. It is obvious that riding comfort are secured rationally with 83.3 percent of road sections are in rather fair condition, and the number of sections in good and very good condition is around 68 percent.

Table 6.11 Pavement conditions - Distribution of Roughness in IRI

Case	Range of IRI (m/km)	Number of Samples	Percentage	Accumulated Percentage	Condition
1	0≤ IRI < 2	4759	**9.69%**	**9.69%**	Very Good
2	2≤ IRI < 4	28719	**58.48%**	**68.17%**	Good
3	4≤ IRI < 6	7431	**15.13%**	**83.30%**	Fair
4	**6≤ IRI < 8**	3210	**6.54%**	**89.83%**	**Poor**
5	**8≤ IRI**	4994	**10.17%**	**100.00%**	**Very Poor**

In comparison with pavement deformation and deflection, cracking is less critical with nearly 60 percent of road section are in very good condition or no occurred crack (Table 6.12).

Table 6.12 Pavement conditions - Distribution of Cracking ratio

Case	Range of Cracking Ratio (%)	Number of Samples	Percentage	Accumulated Percentage	Condition
1	Crack = 0	29432	**59.93%**	**59.93%**	Very Good
2	0< Crack < 10	9269	**18.87%**	**78.80%**	Good
3	10≤ Crack < 20	2759	**5.62%**	**84.42%**	Fair
4	**20≤ Crack < 30**	1499	**3.05%**	**87.47%**	**Poor**
5	**30≤ Crack < 40**	1950	**3.97%**	**91.44%**	**Very Poor**
6	**40≤ Crack**	4204	**8.56%**	**100.00%**	**Serious**

Availability of the three above pavement performance indicators makes it be convenient to calculate MCI value (Eq. 5.4; 5.5; 5.6; and 5.7 in Chapter 5) which is shown in Table 6.13. Based on the distribution of MCI value, 15.8 percent of total road sections belong to the list of candidate repair sections with the value less than 4.

It is natural that the criteria for selection of pavement candidate repair sections based on its performance indicators and index should be customized to fit to the application conditions in reality including consideration of both financial and institutional issues. Based on the criteria in this research, the amount of sections in need of urgent repair in percentage for single distresses of rutting, roughness, cracking and combined distress in MCI are 27.54, 16.7, 15.58, and 15.8, respectively. Such demand is much more higher than the approved work volumes in recent year within the constraint of allocated budget (DRVN, 2012) [12]. And among occurred

distresses on the pavements, rutting is the most critical and becomes very challenging for treatment.

Table 6.13 Pavement conditions - Distribution of MCI

Case	Range of MCI values	Number of Sections	Percentage	Accumulated Percentage	Condition (*)
1	0≤ MCI < 1	1244	**2.53%**	**2.53%**	**Heavily Deteriorated**
2	1≤ MCI < 2	2064	**4.20%**	**6.74%**	
3	2≤ MCI < 3	1920	**3.91%**	**10.64%**	
4	3≤ MCI < 4	2532	**5.16%**	**15.80%**	**Deterioration Progressing**
5	**4≤ MCI < 5**	3371	**6.86%**	**22.66%**	Partly Deteriorated
6	**5≤ MCI < 6**	5539	**11.28%**	**33.94%**	Preferable
7	**6≤ MCI < 7**	12912	**26.29%**	**60.23%**	
8	**7≤ MCI < 8**	14689	**29.91%**	**90.14%**	
9	**8≤ MCI < 9**	4766	**9.70%**	**99.85%**	
10	**9≤ MCI ≤10**	76	**0.15%**	**100.00%**	

Note) * it is referred to practices of PMS in Nagasaki Prefecture, Japan for pavement condition evaluation.

The warning situation of road quality in recent years clearly explains for the high demand for a systematic approach of road management and maintenance to avoid the worst scenario of infrastructure in ruin. Road administrators must not only grasp the current conditions of infrastructure under their jurisdiction but also understand its deterioration or transition of condition states in the future that could be obtained by development of pavement forecasting models and formulation of necessary datasets for proper planning. In broader scope, a PDCA cycle must be indispensable.

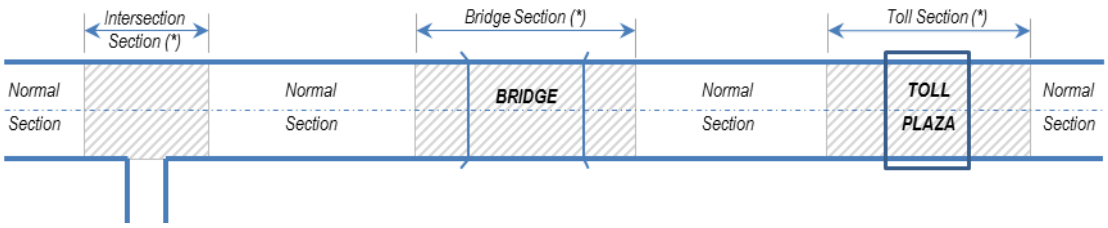
6.4.3 Forecasting Pavement Deterioration by Types of Road Sections

6.4.3.1 Overview

In this empirical study, it was planned to specify pavement deterioration in different traffic conditions as well as on various pavement section types by applying the benchmarking analysis to make proper recommendations for road administrator to take right actions in the whole pavement management PDCA cycle with the focus on making recommendations of improvement the current pavement design standard in Vietnam as the first target.

Similar to empirical study in Chapter 2, the dataset in this study also had been formulated based upon the latest pavement condition survey in 2012 with the coverage of 21 national highways in the northern Vietnam. Preparation of another time-series dataset had been conducted based on maintenance history data with the assumption to set the best condition state for pavement at the completion time of new construction or the latest big repair. In the datasets, pavement samples are defined as the homogeneous sections of 100 meters in length

on one traffic lane for each direction. Of the total number of 47,820 pavement sections for analyzing, five categories in terms of road section types: (1) normal section, (2) bridge approach section, (3) intersection approach section, (4) bridge deck section, and (5) toll approach section have been proposed depend on its physical features and traffic affecting conditions.



Note: (*) including approach sections

Figure 6.7 Layout of typical road facilities and corresponding sections

The objective of benchmarking study is to clarify the needs for applying proper pavement structural solutions for each type or group of road sections. To make evaluation of pavement deterioration, three pavement individual indicators of rutting, roughness in IRI, and cracking have been selected. Beside these three single indicators, overall pavement index of MCI is decided for selection in this study. Definition of ranks or condition states for pavement conditions was shown on Table 6.14 that is also similar to the empirical study in Chapter 2.

Table 6.14 Definition of ranks or condition states for pavement conditions

Rank	Range of pavement indicator or index values				Remark
	Rutting (mm)	IRI (m/km)	Crack ratio (%)	MCI	
1	[0; 10)	[0; 2)	0	(8; 10]	Very good
2	[10; 20)	[2; 4)	(0; 10)	(6; 8]	Good
3	[20; 30)	[4; 6)	[10; 20)	(4; 6]	Fair
4	[30; 40)	[6; 8)	[20; 30)	(2; 4]	Poor
5	[40; 50)	≥ 8	[30; 40)	≤ 2	Very poor
6	≥ 50		≥ 40		Serious

6.4.3.2 Estimation Results and Benchmarking

Pavement deterioration evaluation module analyzes pavement deterioration progress or the transition of pavement condition states or pre-defined ranking. Markov transition probability theory is applied in the analysis to calculate the transition probabilities from a certain ranking to other rankings of pavement deterioration based on periodically observed pavement conditions to specify pavement deterioration rate and its life expectancy in each ranking and in total.

Hazard rate θ_i has been applied to calculate the Markov transition probability π_{ij} in Eqs. 2.24a, 2.24b, 2.24c, 2.24d in Chapter 2 for the transition interval $Z = 1\ year$. Results of Markov transition probability matrixes for Benchmark case are described as follows.

$$\begin{bmatrix} 0.33994 & 0.58861 & 0.06658 & 0.00442 & 0.0004 & 0.00005 \\ 0 & 0.82086 & 0.16223 & 0.01493 & 0.00171 & 0.00027 \\ 0 & 0 & 0.82283 & 0.14711 & 0.02476 & 0.00529 \\ 0 & 0 & 0 & 0.68991 & 0.23096 & 0.07914 \\ 0 & 0 & 0 & 0 & 0.55906 & 0.44094 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

a) Markov transition probability matrix for Rutting distress - Benchmark case

$$\begin{bmatrix} 0.60514 & 0.37514 & 0.01806 & 0.00148 & 0.00019 \\ 0 & 0.9091 & 0.07996 & 0.00937 & 0.00157 \\ 0 & 0 & 0.77275 & 0.17955 & 0.0477 \\ 0 & 0 & 0 & 0.62538 & 0.37462 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

b) Markov transition probability matrix for Roughness distress - Benchmark case

$$\begin{bmatrix} 0.9143 & 0.07717 & 0.00692 & 0.00122 & 0.00029 & 0.00011 \\ 0 & 0.81026 & 0.13806 & 0.03523 & 0.01115 & 0.0053 \\ 0 & 0 & 0.52299 & 0.26278 & 0.12498 & 0.08926 \\ 0 & 0 & 0 & 0.30691 & 0.29835 & 0.39474 \\ 0 & 0 & 0 & 0 & 0.20507 & 0.79493 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

c) Markov transition probability matrix for Cracking distress - Benchmark case

Markov transition probability π_{ii} in the three above matrixes describes different patterns of pavement deterioration among three typical distresses of rutting, roughness and cracking. For rutting, π_{11} is quite small that shows the fast transition from condition state 1 to condition state 2 before the more stable transition afterward. However, in case of cracking, $\pi_{11}\ and\ \pi_{22}$ are very high that describe the long survival time of pavement in condition state 1 and condition state 2 before moving to rapid transition period from condition state 3 to absorbing status at condition state 6.

In the benchmarking study, the estimation results for heterogeneity factor ε^k of individual group k are given in Table 6.15.

Table 6.15 Distribution of Heterogeneity Factors in terms of Section Types

No.	Section Type	Number of sample	Heterogeneity factor (Epsilon ε) for distress			
			Rutting	Roughness	Crack	Combined distress (MCI)
0	Benchmark Case	47820	1	1	1	1
1	Normal	45164	1.01	0.95	1.00	1.00
2	Bridge Approach	1501	0.91	2.11	1.18	1.10
3	Intersection Approach	415	1.04	1.26	0.64	0.88
4	Bridge Deck	688	0.66	2.50	0.71	0.77
5	Toll Approach	52	1.31	2.63	0.62	0.81

The heterogeneity factors ε of individual group are not so much different in rutting and crack but much more different in case of roughness distress that shows wide gap of deterioration speeds in comparison with the Benchmark case.

Once hazard rate is available, expected life expectancy of a condition state i for the road pavement group k (RMD_i^k) or pavement survival time in the condition state is determined as the inversion of hazard rate θ_i^k. As the result, pavement life is estimated as shown in Table 6.16, Table 6.17, Table 6.18 in rutting, roughness, and cracking respectively.

Table 6.16 Estimated Life expectancy of condition states and pavement life in RUTTING

No.	Section Type	Hazard rate θ_i in RUTTING for condition state					Survival time in RUTTING in condition state (years)					Pavement life (years)
		1	2	3	4	5	1	2	3	4	5	
-	Benchmark Case	1.08	0.20	0.20	0.37	0.58	0.93	5.07	5.13	2.69	1.72	**15.53**
1	Normal	1.09	0.20	0.20	0.37	0.59	0.92	5.02	5.08	2.67	1.70	**15.39**
2	Bridge Approach	0.98	0.18	0.18	0.34	0.53	1.02	5.56	5.63	2.96	1.89	**17.06**
3	Intersection Approach	1.12	0.21	0.20	0.39	0.60	0.89	4.88	4.94	2.59	1.66	**14.95**
4	Bridge Deck	0.71	0.13	0.13	0.24	0.38	1.41	7.71	7.80	4.10	2.62	**23.64**
5	Toll Approach	1.41	0.26	0.25	0.48	0.76	0.71	3.88	3.93	2.06	1.32	**11.90**

Table 6.17 Estimated Life expectancy of condition states and pavement life in ROUGHNESS

No.	Section Type	Hazard rate θ_i in ROUGHNESS for condition state				Survival time in ROUGHNESS in condition state (years)				Pavement life (years)
		1	2	3	4	1	2	3	4	
-	Benchmark Case	0.50	0.10	0.26	0.47	1.99	10.49	3.88	2.13	**18.49**
1	Normal	0.48	0.09	0.24	0.44	2.10	11.08	4.09	2.25	**19.52**
2	Bridge Approach	1.06	0.20	0.54	0.99	0.94	4.97	1.84	1.01	**8.76**
3	Intersection Approach	0.63	0.12	0.33	0.59	1.58	8.32	3.08	1.69	**14.66**
4	Bridge Deck	1.26	0.24	0.65	1.18	0.80	4.19	1.55	0.85	**7.39**
5	Toll Approach	1.32	0.25	0.68	1.23	0.76	3.99	1.48	0.81	**7.04**

Table 6.18 Expected Life expectancy of condition states and pavement life in CRACKING

No.	Section Type	Hazard rate θ_i in CRACKING for condition state					Survival time in CRACKING in condition state (years)					Pavement life (years)
		1	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	
-	Benchmark Case	0.09	0.21	0.65	1.18	1.58	11.16	4.75	1.54	0.85	0.63	**18.93**
1	Normal	0.09	0.21	0.65	1.18	1.59	11.14	4.74	1.54	0.85	0.63	**18.90**
2	Bridge Approach	0.10	0.25	0.76	1.38	1.85	9.54	4.06	1.32	0.72	0.54	**16.18**
3	Intersection Approach	0.06	0.13	0.41	0.75	1.01	17.59	7.49	2.43	1.33	0.99	**29.84**
4	Bridge Deck	0.07	0.16	0.49	0.89	1.19	14.89	6.34	2.06	1.13	0.84	**25.26**
5	Toll Approach	0.06	0.13	0.40	0.74	0.99	17.92	7.63	2.48	1.36	1.01	**30.41**

Table 6.19 shows the estimation and calculation results of hazard rate, survival time, and pavement life for all five types of pavement sections in combined distress charactizing by MCI.

Table 6.19 Expected Life expectancy of condition states and pavement life in MCI

No.	Section Type	Hazard rate θ_i in combined index MCI for condition state				Survival time in combined index MCI in condition state (years)				Pavement life (years)
		1	**2**	**3**	**4**	**1**	**2**	**3**	**4**	
-	Benchmark Case	0.58	0.10	0.18	0.15	1.73	9.58	5.64	6.68	**23.63**
1	Normal	0.58	0.10	0.18	0.15	1.73	9.56	5.63	6.67	**23.59**
2	Bridge Approach	0.64	0.12	0.20	0.17	1.57	8.67	5.11	6.05	**21.40**
3	Intersection Approach	0.51	0.09	0.16	0.13	1.96	10.85	6.39	7.57	**26.78**
4	Bridge Deck	0.44	0.08	0.14	0.12	2.25	12.42	7.31	8.66	**30.65**
5	Toll Approach	0.47	0.08	0.14	0.12	2.15	11.87	6.99	8.28	**29.29**

The difference of performance curves or deterioration mechanism, as well as pavement life can be visualized by graphs in Figure 6.8, Figure 6.9, Figure 6.10. Due to the support of concrete slabs under flexible surface of asphalt concrete layers on bridge deck, these sections show better performance in rutting resistance in general. Rutting for toll approach and intersection approach sections that bear repetitive, slow and stationary vehicle loads, and high shear stresses due to transversal forces from vehicle deceleration and acceleration is progressed in more faster speed than others that leads to the biggest loss of 3.6 and 11.7 years of its life cycle in comparison with benchmark case and the best performance case of bridge deck, respectively.

Figure 6.8 shows performance of bridge approach sections in rather fair condition in comparison with benchmark and the worst cases. However, in reality, it can be practically recognized that rutting in bridge approach is always very critical due to the contribution of roadbed consolidation in high embankment that leads to the deep hump effect at the bridge ends to dramatically reduce road riding comfort. Therefore, re-filling rutted and deflected areas becomes common treatment for these sections with much higher frequency in comparison with other sections. And in this case, the data collected in 2012 does not fully reflect real deterioration progress of bridge approach sections.

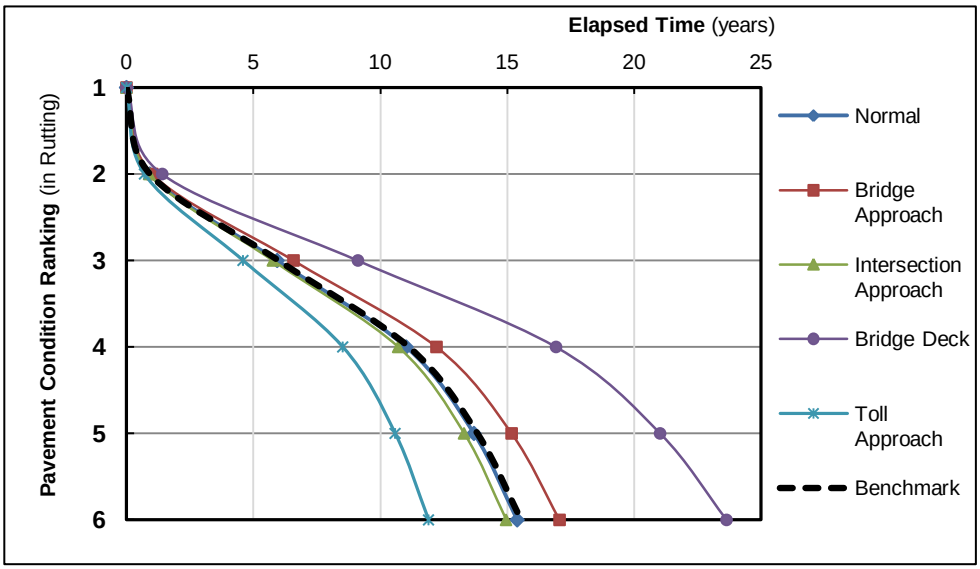


Figure 6.8 Pavement deterioration curves for different section types in maximum rutting

Pavement roughness is the most sensitive performance indicator for all road users that is paid much attention by road administrator, designers, and construction companies also. Along the road, there are road facilities such as bridges, culverts, toll plazas, etc,. located somewhere that create specific operation for vehicles including affecting features to pavement in comparison with normal sections.

Very wide gap between pavement performances in roughness illustrated in Figure 6.9 not only reflects such above specific affecting condition but also requires for different approach in pavement engineering. While the average serviceable life in roughness for all road sections is around 18.5 years, there exists very big loss of pavement life of around 60 percent in toll approach sections and bridge approach sections with its life of just seven and 7.4 years, respectively. As the result, road users have to suffer from heavily deteriorated conditions on these sections that take great contribution to reduction of riding comfort and traffic safety. Such general trend in Figure 6.9 can be understood by highway or pavement engineers but it seems that not so many of them have the image of such quantitative big difference in reality. The real estimated results of pavement performance will certainly request for the significant change in pavement engineering with serious consideration of designing proper pavement structure or applying proper technology for each road section type.

Moreover, the remarkable roughness increasing in bridge sections also makes designers have to seriously take into account for making proposal or recommendation of proper technologies for bridge expansion joints.

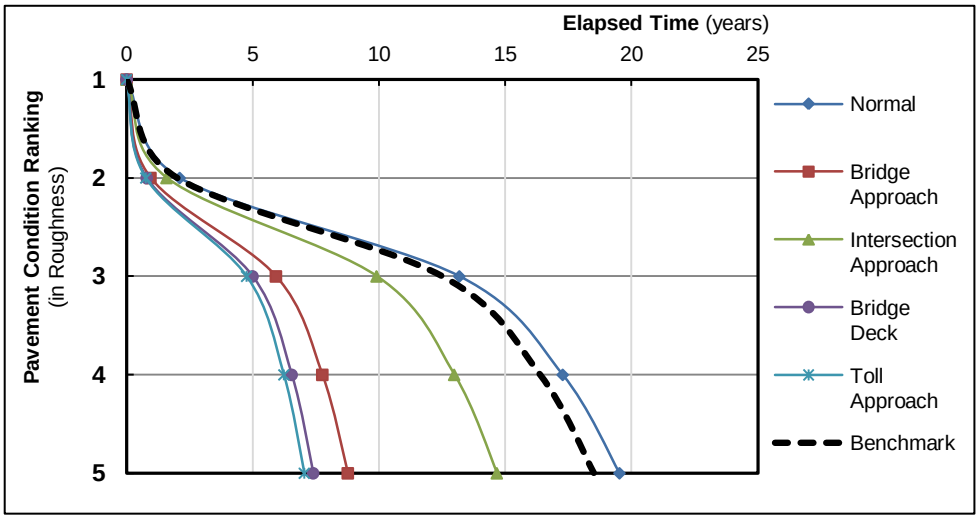


Figure 6.9 Pavement deterioration curves for different section types in roughness

Cracking is one of the typical pavement distresses with strong relation to pavement strength or loading capacity for both flexible and rigid pavements. Similar to benchmarking analysis results for rutting and roughness indicators, performance curves in cracking for five pavement section types are also very much different in a wide range as shown in Figure 6.10. Bridge approach section is the most critical case with highest deterioration speed and shortest service life of 16.18 years, around nine years shorter than bridge deck section.

Figure 6.10 shows the best cases with longest pavement life cycle in term of cracking on intersection approach and toll approach sections that is totally opposite to the results of rutting and roughness performance in Figure 6.8 and Figure 6.9. There seems to exist some wrong understanding or improper explanation for system analyzers who are not senior in pavement engineering in this case. However, from engineering consideration, it can be seen that the benchmarking analysis results for pavement indicators of rutting, roughness, and cracking in the empirical study are logical and explanatory. Among these three indicators, rutting and roughness reflect stability of asphalt concrete with very high sensitiveness to high temperature. The more unstable asphalt concrete, the less elastic modulus or rigidity that reduces material deflection and deformation resistance and also increases bending capacity that prevents pavement from cracking. In the worst case, bleeding can be visualized on pavement surface, pavement seriously deteriorates with high rutting depth and critical roughness but no sign of cracking. Therefore, long performance in cracking resistance of intersection approach and toll approach sections in Figure 6.10 under the fact of destructive deformation is meaningless to make evaluation of pavement quality or its life.

Moreover, composite structure of flexible surface of asphalt concrete layers on rigid concrete slab seems to be the reasonable solution for toll sections in term of securing sufficient strength or loading capacity under specialized traffic conditions. However, pavement performance on the section is very poor (Figure 6.8, Figure 6.9) due to the instability of thin overlay asphalt layers and slippery contact area between these two structures. Such benchmarking analysis

should be studied and considered seriously by pavement engineers and road administrator for proper solutions. Among various proposals, it is recommended to use high performance cement concrete for toll sections without asphalt concrete overlaying.

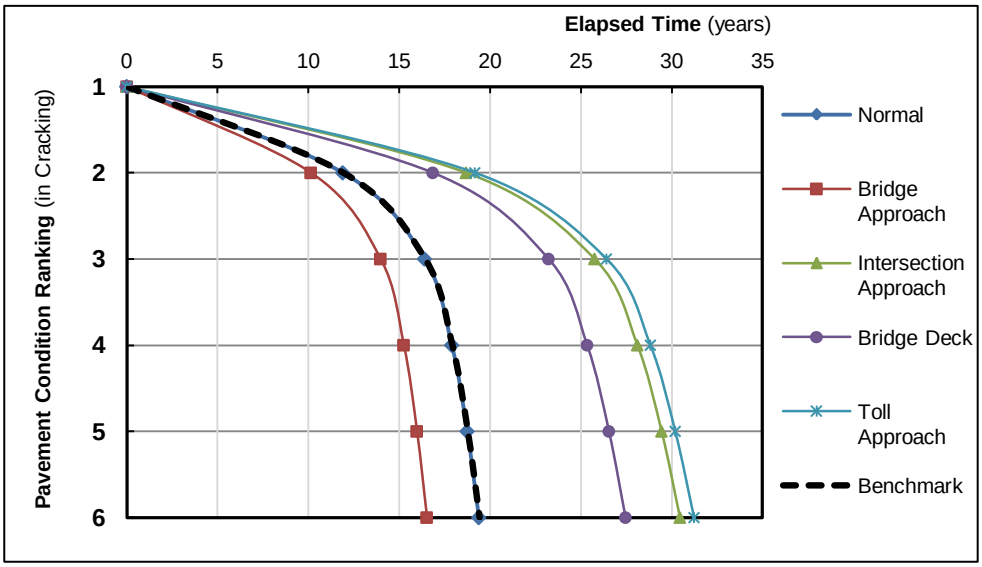


Figure 6.10 Pavement deterioration curves for different section types in crack (2)

Beside the accelerated path of pavement performance curves in Figure 6.8, Figure 6.9 initiating from condition state 4 for rutting and condition state 3 for roughness, the curves also show another early accelerated phase right after putting pavement into services till approaching condition state 2 of rutting in 10mm depth and roughness in 2m/km of IRI. This can be explained for one typical mechanism of rutting and deformation formulation due to insufficient compaction during construction. Right after putting into operation, under vehicles' loads on insufficient density pavement, rutting had occurred as consequence of re-compaction effect. Even such permanent deformation is quite small but it is also one firm warning for road administrator and contractors to pay sufficient attention on securing compaction quality in road construction.

While in individual distress, there exist the big difference among deterioration curves or speeds of pavement section types but such gap is getting narrower in case of combined distress in MCI as shown in Figure 6.11. Therefore, it should be improper to make evaluation of pavement condition or its soundness just based on overall index like MCI. For such purpose, the other typical indicators describing pavement distresses must be selected for analysis.

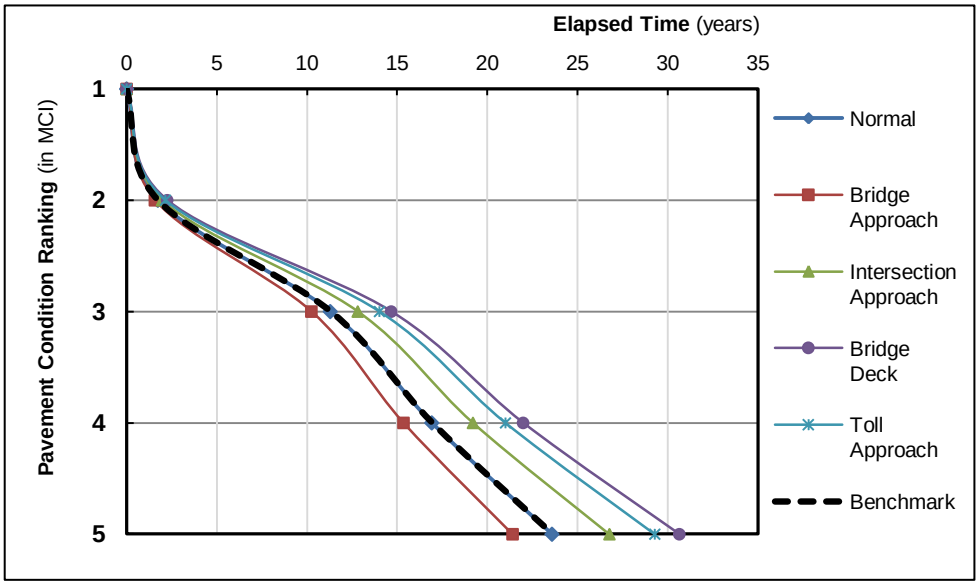


Figure 6.11 Pavement deterioration curves for different section types in combined index MCI

6.4.4 Forecasting Pavement Deterioration by Types of Road Lanes

In previous empirical study, difference of pavement deterioration along the roads has been examined quantitatively. Such difference in pavement deterioration mechanism caused by vehicle loadings not only occurs in longitudinal direction but also in transversal one between adjacent road lanes especially in case of heavy traffic lanes for trucks and light ones for passenger cars. Figure 6.12 shows one typical multi-lane road, National Highway 5 connecting Hanoi Capital and Hai Phong big seaport, with central median strip between two traffic directions. There are three lanes for each direction: the outer lane for non-motorized traffic including motorbikes, and the other inner two lanes for vehicles. Between two vehicle lanes, drivers have to follow the regulation of left-side passing due to the law of left-side steering in Traffic Law that makes the central lane be occupied mostly by slow and heavy vehicles; the lane adjacent to central median strip is mainly occupied by faster car or light vehicles.

Under such typical loading conditions, difference in pavement deterioration between the two vehicle lanes is recognized. And that is the background of this study to quantitatively examine such difference in depth applying benchmarking evaluation and local mixture Markov deterioration hazard model.

The same dataset of pavement conditions as in previous empirical study is kept using in this case. With the desire of quantitative analysis the pavement deterioration under different traffic duties, all these total number of 47,820 pavement sections have been divided into two groups in terms of lane type: (1) mostly light vehicle operation sections, and (2) mixed vehicle operation sections. In road cross-section with multiple-lanes for each direction, the high speed lane for car or mostly light traffic that is adjacent to road median trip belongs to Group 1. On

the other hand, Group 2 consists of all other remaining lanes that could be heavy vehicle lanes on multi-lane roads or mixed lane for all vehicles in the case of roads with only one vehicle lane for each direction. Figure 6.12 is an informative illustration.

Figure 6.12 Typical pavement distresses on traffic lanes

In this study, selection of pavement individual indicators, overall index, and its definitions are totally same with previous study in Section 6.4.3 on types of road sections.

Due to the same dataset application, regarding to the Benchmark case, Markov transition probability π_{ij} is totally similar to the previous study shown in Markov transition matrixes (a), (b), (c) in Section 6.4.3.2.

In the benchmarking study, the estimation results for heterogeneity factor ε^k of individual group k are given in Table 6.20.

Table 6.20 Distribution of heterogeneity factor in terms of lane types

No.	Section Type	Number of sample	Heterogeneity factor (Epsilon ε) for distress			
			Rutting	Roughness	Crack	Combined distress (MCI)
0	Benchmark	47820	1	1	1	1
1	Mostly light vehicle operation	17608	0.73	0.62	0.43	0.70
2	Mixed vehicle operation	30212	1.19	1.32	1.39	1.21

Regarding all single pavement distresses of rutting, roughness and cracking, the heterogeneity factors ε of Group 2 is much more higher than that value of Group 1. The most critical case is

cracking distress with ε of Group 2 is three times higher than that value of Group 1 that makes so big gap between their performance curves.

Results of benchmarking analysis for comparative view of the deterioration progress of different pavement types and groups including benchmark case are shown in Table 6.21, Table 6.22, Table 6.23 based upon calculated values of hazard rate θ_i^k and life expectancy for each condition state i for all cases. Pavement deterioration curves in terms of rutting, roughness, and cracking were created that visually shows the transition of pavement condition states from the best condition to the worst one through its service life.

Table 6.21 Life expectancy of condition states and pavement life in rutting

No.	Lane Type	Hazard rate θ_i in RUTTING for condition state					Survival time in RUTTING in condition state (years)					Pavement life (years)
		1	2	3	4	5	1	2	3	4	5	
0	Benchmark Case	1.08	0.20	0.20	0.37	0.58	0.93	5.07	5.13	2.69	1.72	**15.53**
1	Mostly light vehicle operation	0.79	0.14	0.14	0.27	0.42	1.27	6.96	7.05	3.70	2.36	**21.35**
2	Mixed vehicle operation	1.29	0.24	0.23	0.44	0.69	0.78	4.25	4.30	2.26	1.44	**13.04**

Based on the criteria for pavement condition in term of rutting as specified in Table 6.14, the estimated result shows the average serviceable life for asphalt pavements on road in the northern Vietnam of 15.53 years.

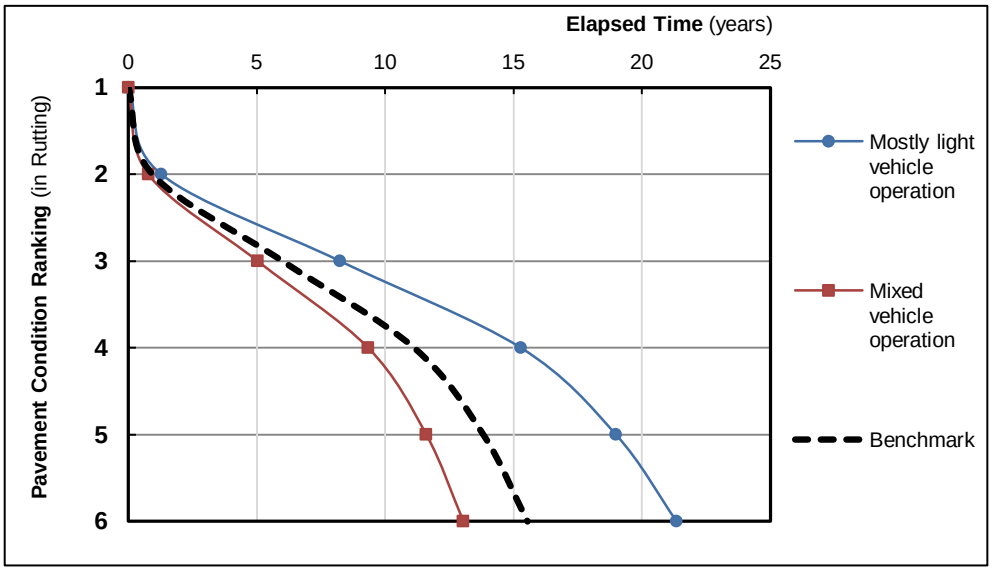


Figure 6.13 Pavement deterioration curves in maximum rutting due to light and heavy duty traffic

There is a big difference in pavement service lifes of 8.31 years between light traffic lanes and mixed traffic lanes that equals to 39 percent of loss in service life of mixed traffic lanes in

comparison with light ones due to the major cause of heavy vehicle operation including overloaded trucks.

Estimation results of pavement deterioration in roughness progression are shown in Table 6.22 and Figure 6.14.

Table 6.22 Life expectancy of condition states and pavement life in roughness

No.	Lane Type	Hazard rate θ_i in ROUGHNESS for condition state				Survival time in ROUGHNESS in condition state (years)				Pavement life (years)
		1	2	3	4	1	2	3	4	
0	Benchmark Case	0.50	0.10	0.26	0.47	1.99	10.49	3.88	2.13	**18.49**
1	Mostly light vehicle operation	0.31	0.06	0.16	0.29	3.21	16.93	6.26	3.44	**29.84**
2	Mixed vehicle operation	0.66	0.13	0.34	0.62	1.51	7.95	2.94	1.61	**14.00**

Regarding to pavement performance, its roughness is the most sensitive indicator for all road users that is paid much attention by road administrator, designers, and construction companies also.

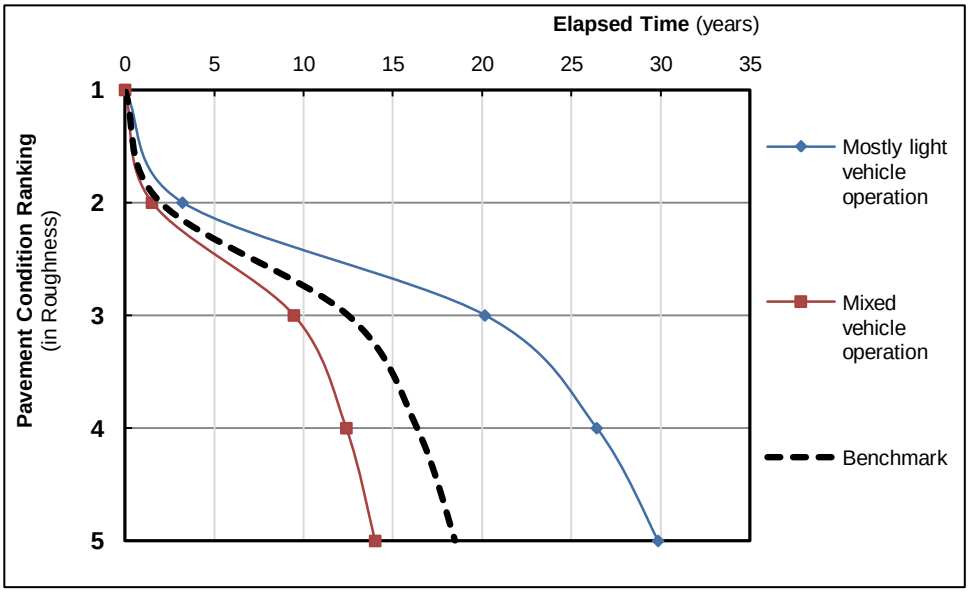


Figure 6.14 Pavement deterioration curves in roughness due to light and heavy duty traffic

The gap between pavement performances of light traffic lanes and mixed ones in roughness is more critical as illustration in Figure 6.14. While the average serviceable life in roughness for all road sections is around 18.49 years, there exists very big loss of pavement life of around 53 percent of mixed traffic lanes in comparison with light ones. As the result, road users have to suffer from heavily deteriorated conditions on mixed traffic lanes that take great contribution to reduction of riding comfort and traffic safety. The real estimated results of pavement performance will certainly request for the significant change in pavement engineering with serious consideration of designing proper pavement structure or applying proper technology for each type of road lanes.

Table 6.23 and Figure 6.15 show the estimation results of pavement deterioration in cracking progression.

Table 6.23 Life expectancy of condition states and pavement life in cracking

No.	Lane Type	Hazard rate θ_i in CRACKING for condition state					Survival time in CRACKING in condition state (years)					Pavement life (years)
		1	2	3	4	5	1	2	3	4	5	
0	Benchmark Case	0.09	0.21	0.65	1.18	1.58	11.16	4.75	1.54	0.85	0.63	**18.93**
1	Mostly light vehicle operation	0.04	0.09	0.28	0.51	0.68	25.94	11.05	3.59	1.97	1.47	**44.01**
2	Mixed vehicle operation	0.12	0.29	0.90	1.64	2.20	8.04	3.42	1.11	0.61	0.45	**13.64**

Similar to benchmarking analysis results for rutting and roughness indicators, performance curves in cracking for the two types of pavement lanes also very much different in a wide range in Figure 6.15. In this case, pavement failure on mixed traffic lanes also is the much more critical with faster deterioration speed and the short service life of 13.64 years, 30.37 years shorter than light traffic lane's corresponding of 69 percent of loss in pavement service life. The photo in Figure 6.12 is one typical example of pavement cracking distress on mostly light traffic lane and mixed traffic lane that can be used to make right explanation for these estimated results.

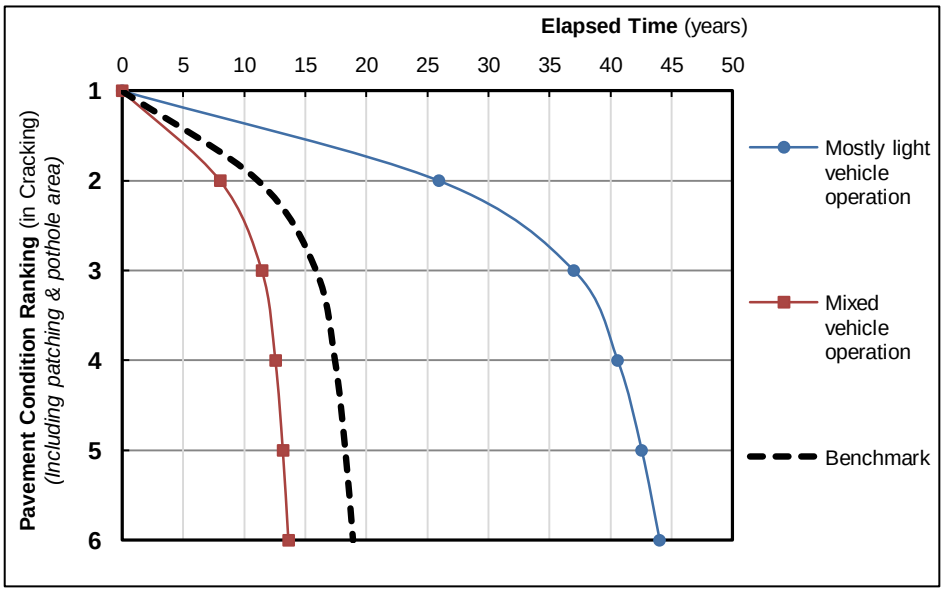


Figure 6.15 Pavement deterioration curves in crack due to light and heavy duty traffic

Beside the accelerated path of pavement performance curves in Figure 6.13 and Figure 6.14 initiating from condition state 4 for rutting and condition state 3 for roughness, the curves also show another early accelerated one right after putting pavement into services till approaching condition state 2 of rutting in 10mm depth and roughness in 2m/km of IRI. This problem is

also similar to the previous study on types of pavement sections that requires for further attention on securing compaction quality in road construction.

Examination of pavement combined distress by introduction of MCI is also conducted. Resuls of estimation and calculation are shown in Table 6.24 and Figure 6.16. The difference between deterioration speeds of pavement Group 1 and Group 2 can be seen but less critical than the individual distress.

Table 6.24 Expected Life expectancy of condition states and pavement life in MCI

No.	Lane Type	Hazard rate θ_i in combined index MCI for condition state				Survival time in combined index MCI in condition state (years)				Pavement life (years)
		1	2	3	4	1	2	3	4	
0	Benchmark Case	0.58	0.10	0.18	0.15	1.73	9.58	5.64	6.68	**23.63**
1	Mostly light vehicle operation	0.40	0.07	0.12	0.10	2.47	13.66	8.04	9.53	**33.70**
2	Mixed vehicle operation	0.70	0.13	0.21	0.18	1.43	7.93	4.67	5.53	**19.56**

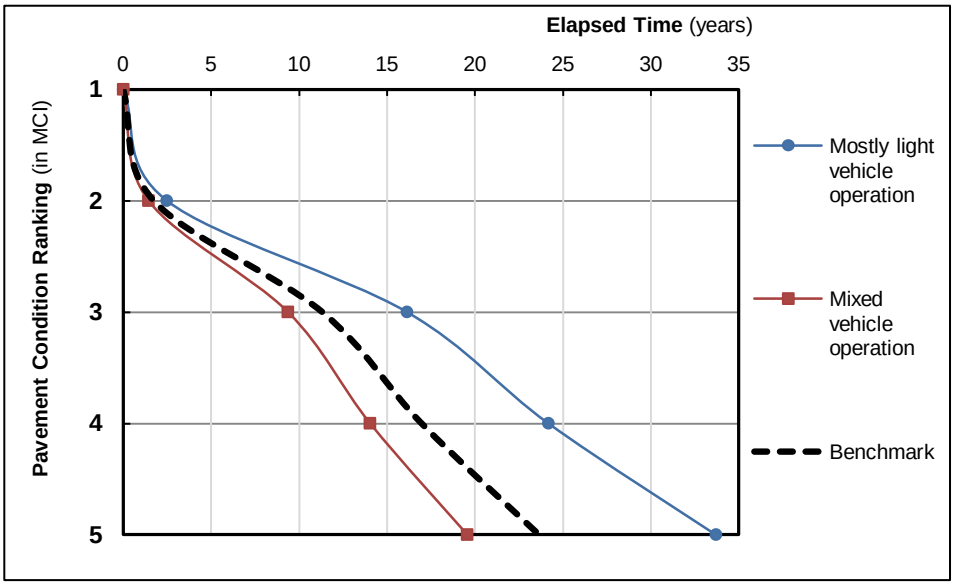


Figure 6.16 Pavement deterioration curves in combined index MCI due to light and heavy duty traffic

6.5 CONCLUSION AND RECOMMENDATIONS

Post-evaluation in pavement management and maintenance is very significant that best uses all available data to make innovation for relevant activities in the PDCA cycle. At the very early effort of implementation of the data oriented pavement management system, PMS Kyoto model, outcomes are remarkable and becomes feasible for some improvement of PDCA cycle.

Followings are the key recommendations for consideration to improve the current pavement design standards in Vietnam based on post-evaluation.

6.5.1 Pavement Design Reliability

In comparison with the previous pavement design standard 22TCN 211-93, there is one new point in the current standard 22TCN 211-06 in term of taking into account of the pavement design reliability. In principle, the higher reliability level will be selected in designing pavements of the more significant road.

The most significance of introduction reliability is to eliminate the previous practice of applying unique pavement structure for all sections along the long roads. The results from the first empirical study in this chapter obviously prove the necessity of such important regulation. The concept of pavement design reliability had been introduced in other pavement design manuals or guidelines (ex, AASHTO's Guideline for pavement design).

Regarding to application of the reliability, the current design standard 22TCN 211-06 also regulates three critical sections along the road must have reliability level to be at least one level higher than normal sections on the road: (i) high supervision rate, (ii) toll plaza, (iii) stopping and parking areas. However, based on the above empirical studies, it is strongly recommended to supplement some more cases for application of higher reliability level or better pavement structure in comparision with the normal section also. Two priority types of sections are: (i) bridge approach sections, and (ii) bridge decks.

Moreover, the application of higher reliability level for each type of critical stages in terms of pavement capacity *(critical rebound deformation, plastic deformation in materials, structural cracking due to bending)* can be different each other.

6.5.2 Pavement Structures for Lanes in Multi-lane Road

Both the actual situation of pavement conditions (Figure 6.12) and the forecasted pavement deterioration curves (Figure 6.13, Figure 6.14, Figure 6.15) shows the big challenging for prolonging pavement life because of the large accumulation of deteriorated sections and its life is quite short especially in critical cases.

The problem of too much difference in quality or deterioration status of adjacent lanes in the same road direction on multi-lane roads leads to difficulties in pavement maintenance that may contribute to increase of pavement life cycle cost and vehicle operation cost also. Among many negative influences of such situation, it is understood that reduction of traffic safety or remarkable increase of accident rate is the extreme consequence due to the tendency of vehicles' lane changing. There occurs new behavior of heavy vehicles' drivers of trying to

escape deteriorated lanes and occupy other adjacently better lanes that are mainly served for light vehicle or non-motorization as their violation of traffic law (Figure 6.12).

Regarding to maintenance of deteriorated lanes in this case, pavement cutting and replacement is the most proper countermeasure under the constraint of remaining adjacent lanes in good or better conditions. In comparison with overlaying measure, the selection of pavement cutting and replacement becomes much more costly that limits the candidate repair sections or increase the accumulation of heavily deteriorated sections under budget constraint.

In reality of current pavement practices on design in Vietnam, such situation has been considered but insufficient. As shown in Table 6.1, lane distribution factor f_{lane}^{crit} is introduced in the current pavement design standard 22 TCN 211-06 to determine the design axle loads *(the equivalent number of repetitions of a standard axle load per day per design lane at the last year of pavement designed life)* in the design lane. So the design lane is the most critical lane in term of traffic loading. Then designed pavement structure of design lane is applied for all motorized lanes in each direction and in the whole cross-section of road carriageway. And that is the main cause for difference in term of pavement deteriorated conditions of lanes on multi-lane roads.

Therefore, it is recommended to change such regulation in the current pavement design standard 22 TCN 211-06. On the roads with multi-lane in each direction, pavement for the most critical lanes will be designed with the critical lane distribution factor f_{lane}^{crit}. However, for other remaining lanes, its actual lane distribution factor f_{lane} should be applied instead of the critical one. As the result, there can be difference of pavement structure for adjacent lanes as shown in Figure 6.17.

It can be seen that the above proposal of pavement structures is quite rare in reality due to the inconvenience in construction. To make the recommendation becomes more practical, another option of pavement structure in Figure 6.18 can be the proper proposal. In this proposal, material property is the target for changing pavement loading capacity rather than changing thickness of pavement layers to secure for pavement construction convenience. Using modified polimer asphalt concrete is one of the proposal. Various researches have pointed out that modified polimer asphalt is high performance asphalt that can improve both rutting resistance and crack resistance or fatigue cracking. Further research on using chemical substance or additives to improve characteristics and performance of asphalt concrete is also strongly encouraged as the intermediate solution of plant-mix technology before approaching to pre-mix technology. Among various additives for asphalt that had been studied intensively in Japan since 1950's, SBS (Styrene-Butadiene-Styrene) has been recognized as the best additive for enhancing performance and quality of asphalt concrete. Since 1988, modified polimer asphalt in pre-mix technology has become popular in Japan for road pavement construction especially for trunk roads [14].

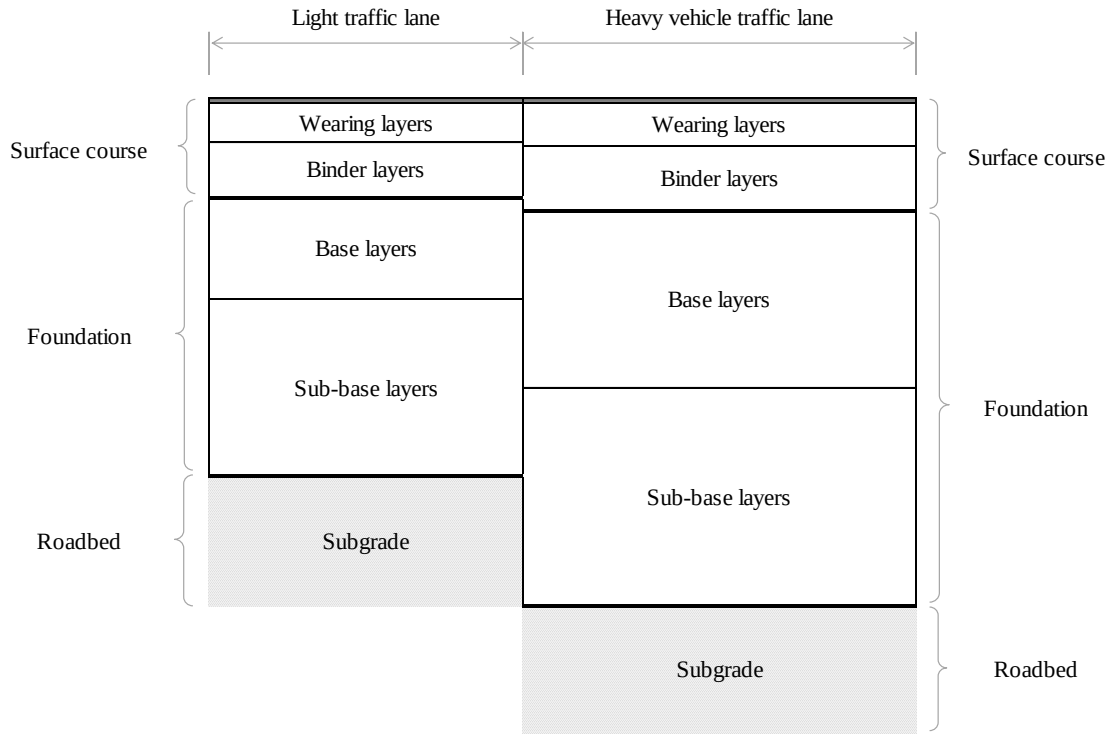


Figure 6.17 Pavement structures for light traffic lane and adjacent heavy vehicle traffic lane

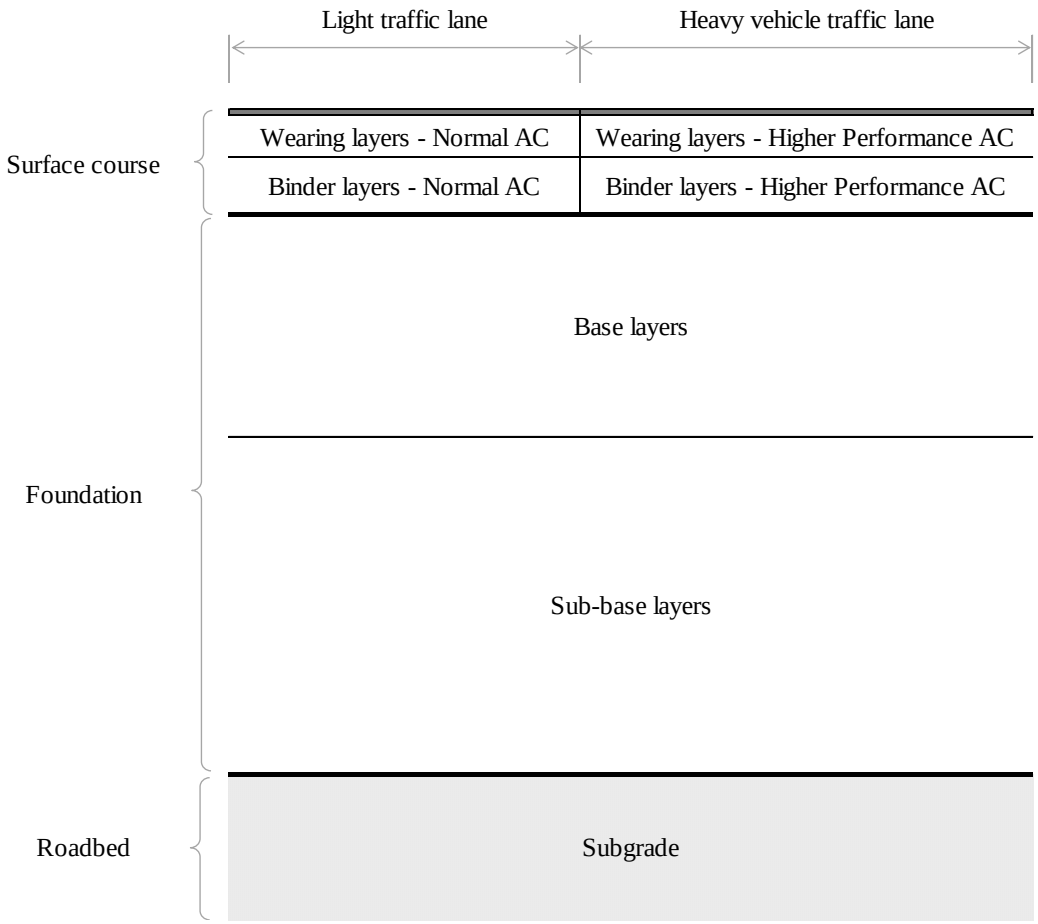


Figure 6.18 Pavement structures for light traffic lane and adjacent heavy vehicle traffic lane

Empirical study had been conducted based on the models and pavement datasets of its conditions in national roads in the northern Vietnam. Two pavement categories in terms of traffic lane types have been formulated based on the data to clarify its difference in pavement performance and deterioration. In reality, mostly same pavement structure had been designed to apply for all traffic lanes on the road cross-section. Estimated results of two corresponding pavement performance curves in terms of the three pavement indicators show big gaps of pavement life of more than three times in the maximum difference case. It becomes so critical that make road administrator and engineers indispensably take serious consideration for remarkable change in pavement engineering for lengthening its life and securing road serviceability. Under the situation, it is strongly recommended to apply different pavement structures for different lanes in road cross-section (Figure 6.18) especially in current national economic even it can be recognized that there will be some difficulties for construction works.

6.5.3 Pavement Performance Models

Structural response models are introduced in the current pavement design standard 22 TCN 211-06 to compute critical pavement rebound deformation, maximum shearing stress in the subgrade and unbound materials, maximum tensile bending stress at the bottom of cementitious material layers due to traffic loads at the critical condition in term of the highest number of ESAL in the last year of design life. The concept of cummulative number of ESAL has not been applied that leads to the lack of some significant models of accumulated damages including performance models.

Permanent deformation in asphalt mixtures has not been taken into account that makes the rutting distress can not be controled in pavement design. Results of the above empirical studies brings the deep anxiety to road administrator and engineers on pavement distresses especially rutting or pernanent deformation. Therefore, it is also strongly recommended to supplement performance models taking into consideration of accumulated traffic loads. Rutting model must be the first priority.

Following paragraphs are the description of permanent deformation model of the AASHTO Mechanistic-Empirical Pavement Design Guide for asphalt bound and unbound layers as a reference.

Combination of repeated load permanent deformation test in the laboratory and statistical analysis of field data for callibration had been made to propose rutting model in the asphalt mixtures as follow (F.T.Fwa, 2006) [2].

$$\frac{\varepsilon_p}{\varepsilon_r} = k_1 \times 10^{-3.4488} \times T^{1.5606} \times N^{0.479244} \tag{6.9}$$

where

- ε_p : accumulated plastic strain at N repetitions of axle load (in./in.)
- ε_r : resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in./in.)
- N: number of load repetitions
- T: mixing temperature (deg F).
- Cofficient k_1 can be calculated from following equations

$$k_1 = (C_1 + C_2 \times D) \times 0.328196^D$$

$$C_1 = -0.1039 \times H_{ac}^2 + 2.4868 \times H_{ac} - 17.342$$

$$C_2 = 0.0172 \times H_{ac}^2 - 1.7331 \times H_{ac} + 27.428$$

where

- H_{ac} : the total thickness of the asphalt layers, in.
- D : depth below the surface, in.

Equation 6.10 shows the permanent deformation model for the unbound layers of granular base or sub-base after N axle load repetitions in flexible pavement structure.

$$\delta_a(N) = 1.673 \times (\frac{\varepsilon_0}{\varepsilon_r}) \times e^{-(\frac{\rho}{N})^\beta} \times \varepsilon_v \times h \tag{6.10}$$

where

- ε_0, β, ρ : material properties
- ε_r : resilient strain imposed in laboratory test to obtain material properties ε_0, β, ρ
- ε_v : average vertical resilient strain in the unbound layers as obtained from the primary response model.
- h : thickness of the unbound layers, in.

$$Log\ \beta = -0.61119 - 0.017638 \times W_c$$

$$\rho = 10^9 \times \left\{ \frac{C_0}{[1 - (10^9)^\beta]} \right\}^{\frac{1}{\beta}}$$

$$C_0 = Ln(\frac{a_1 \times M_r^{b_1}}{a_9 \times M_r^{b_9}})$$

where

- W_c : water content, %
- M_r : resilient modulus of the unbound layer or sublayer, psi
- $a_1,\ a_9$: regression constants
- $b_1,\ b_9$: regression constants

The calibrated model of permanent deformation for all subgrade soil is proposed as follows:

$$\delta_a(N) = 1.35 \times (\frac{\varepsilon_0}{\varepsilon_r}) \times e^{-(\frac{\rho}{N})^{\beta}} \times \varepsilon_v \times h \qquad (6.11)$$

Finally, the total rutting in the flexible pavement structure is the summation of all individual permanent deformation in asphalt layers, foundation of granular base and subbase, and subgrade as shown in Eq. (6.9, 6.10, and 6.11).

AASHTO Mechanistic-Empirical Pavement Design Guide also proposed the other models for pavement performance in roughness and cracking *(alligator cracking, longitudinal cracking, transverse cracking)*. In the reality of application, there are many further empirical studies in Departments of Transportation in America for customization the models [13].

Structural response models are the crucial component of mechanistic methods for pavement design including current pavement design standard in Vietnam. Recommendation for supplementation of performance model of rutting to the standard is just the priority. In the broaden understanding, incorporating empirical procedure into the originally mechanistic method to improve practices of pavement design by formulation a new method of mechanistic-empirical in Vietnam should be considered seriously. Implementation of pavement PDCA cycle in the U.S. for several decades since AASHO Road Test in 1950's had pointed out the limitations of their original pavement design guide (AASHTO 1993) in comparison with the new situation that required for the improvement by incorporating mechanistic procedure to empirical model to develop AASHTO MEPDG in 2004. There seems to exist some suggession for improvement of pavement design standard in Vietnam based on such example.

More over, in the current pavement design standard in Vietnam, the second critical state to examine possibility of plastic deformation in pavement materials (equation 6.6) should be revised. Not only subgrade and unbound materials are the target for examining but also asphalt concrete layers need being verified. Because there is one patern of rutting distress in the form of plastic deformation in asphalt concrete layers caused by extreme shearing stress that is very common in reality in many roads in Vietnam recently.

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CHAPTER 7

Conclusions

7.1 BRIEF SUMMARY

The research in this dissertation consists of seven chapters presenting implementation of pavement management system in Vietnam in two main parts of the contents.

In the first part of this dissertation, beside the general introduction and problem statement to clarity the necessity of the research in Chapter 1, the main content focuses on theory study of deterioration forecasting models, PMS Kyoto model and their application in empirical studies. Markov hazard model and Local mixture Markov hazard model were intensively studied in Chapter 2. Results from empirical studies prove that approaching to stochastic forecasting models is proper and useful for pavement management and maintenance planning without serious burden on data work. Especially, Benchmarking analysis in the Local mixture Markov hazard model is recognized as an powerful toll to determine heterogeneity factor ε that exists in each group of pavements to characterize their comparative speeds of deterioration.

There is no doubt about great effects from benchmarking analysis because it can quantitatively point out the critical groups of sections with fast or abnormal deterioration that need being investigated further to specify the causes and proper treatments. For pavements, intensive and structural surveys of loading capacity like Falling Weight Deflection test should be performed just for these candidate sections instead of doing tests for all sections including many sections in good condition. The principle of classification of data items into different hierarchical levels for utilization and collection has been realized by applying benchmarking analysis that supports for optimization of data collection in both terms of cost and time.

Beside Markov hazard model and Local mixture Markov hazard model, Chapter 3 also presents the study on Hidden Markov model and its application to examine mutual relation between road surface deterioration and decrease in pavement load bearing capacity. This sophisticated model is expected for the next step of upgrading PMS in the roadmap of PMS development and implementation when database will become richer.

Chapter 4 focuses on studying a new PMS named as “Kyoto model” for pavement management which employs Markov hazard model and Local mixture Markov hazard model. Understanding the structure, components of Kyoto model and its functions is necessary for

development a new PMS system in Vietnam from system architecture design to formulation of calculation flows. Structure of Kyoto model and its components best fit to system functions supporting for the implementation of PDCA cycle.

The main research in the second part of this dissertation focuses on development one new PMS system in Vietnam based on Kyoto model as a platform, and operation the new system. The first part in Chapter 5 summarizes practices of road management and history of application of PMS systems and lessons learned in Vietnam.

More than one decade of applying ready-made PMS systems till 2010 with huge investment and allocated resources but poor outcomes, serious consideration had been made by road administrators in Vietnam to decide the direction. Finally, the decision had been made to choose the direction of development a new PMS system that fits to the local conditions in Vietnam and the possibility for upgrading in the future based on increasing demand. In order to make such decision, reviewing all relevant factors had been conducted and discussed with the confirmation of some key points as follows:

- Global trends in PMS: one-size-fits-all systems are incapable of dealing with demands for customization that differ from country to country. One-finds-one's-own-size systems seem to be the best solution. And some countries had initiated this new approach.
- Academic achievements in the field of infrastructure asset management in Japan and dissemination to Vietnam.

  With remarkable achievements in development of infrastructure stock, great attention has been paid to asset management and maintenance in Japan including many researches on asset management systems mainly started in the early 21st century. Stochastic deterioration models using Markov theory had been intensively studied for application instead of conventionally deterministic models. Many sophisticated papers have been published since the 2000s to share and disseminate new approaches. The The new pavement management system named as Kyoto model developed within the framework of academic research program is one typical academic achievements in Japan to make a solid background for implementation in reality.

  However, it is crucially important that these academic achievements had been no longer the domestic value. It has been comprehensively disseminated to Vietnam for years through variety of trainings within framework of collaboration. It is confirmed that not only engineers and road practitioners but also many researchers and university lecturers in Vietnam have the chance to know and understand new approaches and knowledge in the field of road asset management including introduction of and their understanding of the Kyoto model. Much new information has been incorporated into

university lectures to disseminate to students that secures for a firm step of human resource preparation.

Within the collaboration framework, some researches and case studies on pavement and prediction deterioration models in Vietnam had been conducted with significant results presented to many road administrators and authorities.

For the road sector in Vietnam, their approach to the Kyoto model is at the extremely critical threshold of changing direction in PMS system. However, it is understood that the more important point for successful implementation of a PMS system in Vietnam is the numerous voluntary-based efforts by Japanese researchers with the aim of enhancing human capacity as the most fundamental preparation for any development.

The final part in Chapter 5 describes the full and concrete procedures for development the new PMS in Vietnam and detail description on its functions, component modules including operation and computation flow of each module are discussed in the chapter. The description is expected as the good and useful reference for expanding the development of management systems for other facilities from pavement or to other countries especially in Asian region.

At the end of the second part of this dissertation, initial implementation of PDCA cycle on road pavement based on empirical study applying the newly developed PMS is described in Chapter 6 as one step to verify the new system and also the good motivation for sustainable implementation of PMS in Vietnam.

It is clear that PMS system integrated with mathematical models helps to clarify how pavement deteriorates whilst the question of why such deterioration occurs can be answered within the knowledge of pavement engineering. That's common combination in pavement asset management.

7.2 CONCLUSIONS

The goal and objectives of this research on pavement management system implementation in Vietnam have been obtained that presents the backgrounds, history, new approach including methodology and intensive description, current practices and achievement of the implementation. The significant outcomes have come out even earlier than expectation. Main conclusions are summarized as follows:

- Long history in the past with lessons learned of practices on commercial pavement management systems in Vietnam shows that among three approaches for PMS (use commercial software, modify existing commercial software by requesting to developers, and newly develop a customized system), the third option is the best

approach to dynamically satisfy local conditions and specialized requirements that may differ case by case, time to time, region by region.

- Formulation of PMS dataset should be improved in both methodology and technology. It is obvious that database is very significant for any PMS system. However, it is not a simple task to complete a database in a short time under limited budget source. Therefore, there must be proper strategy for database formulation and enrichment taking full advantage of all available inventory data even simple one like pavement history information, and all time-series data. The strategy is mainly decided by selection of PMS system.

  Kyoto model is one of the best systems that make benefit and create motivation for road administrators because it eases or eliminates their burden in data preparation. With very initial effort in data work (in the least case, three data items in the dataset: previous condition state, current condition state, time interval), performance curves of pavement deterioration can be forecasted for making decision in asset management. Moreover, benchmarking analysis in Kyoto model is an extremely useful function to optimize data collection or pavement condition survey.

  Transferring of automatic pavement condition survey technology is also one key factor for simplifying data preparation and successful implementation of pavement management system in Vietnam.
- Among variety of factors related to development and operation of PMS, human capacity and their motivation play the utmost important role to secure for the success and sustainable implementation. Provision of knowledge and technologies to road practitioners is indispensable. In the case that R/D function is immature, collaboration with academic institutions is the good scheme for human capacity enhancement. It is confirmed that the successful implementation of PMS in Vietnam so far is originated and secured by the closed collaboration between Japan and Vietnam at governmental level in general and at academic level between universities in particular.
- Implementation of PDCA cycle is very beneficial because any relevant activity can be optimized to improve pavement management and maintenance continuously that may increase pavement longevity, minimize its LCC, and strengthen accountability to road users and public. In this research, initial implementation of PDCA cycle on road pavement based on empirical study applying useful tool of the new Kyoto model-based PMS points out critical problems in the current pavement design standard need being improved.
- Empirical studies in all four chapters from Chapter 2 to Chapter 6 show the good operability and performance of Markov hazard model, Local mixture Markov hazard model, and Hidden Markov model that are employed in Kyoto model.
- The general roadmap and detail flows of implementation to develop a new customized PMS system can be expanded to infrastructures other than pavement such as bridges, tunnels, airports, etc. in Vietnam or other Asian and developing countries.

For extending study in the future, there will be variety of directions or topics of researches focusing on customization and modification of current pavement practices in Vietnam. The promising and in high priority topic should be intensive study to examine the effects of climatic conditions like ambient temperature, rainfall or solar radiation to pavement deterioration to supplement proper coefficients taking into account of natural conditions in pavement design, construction and maintenance as well. The study will be very significant for one-dimensional, stretched-out country like Vietnam and can be handled by applying Markov hazard model and Local mixture Markov hazard model with more effort on data work.