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IMPROVEMENT OF A PAVEMENT MANAGEMENT SYSTEM INCORPORTAING FLOODING

MISBAH UDDIN KHAN

M.Phil., M.Sc. Eng. (Civil), B.Sc. Eng. (Civil), CPEng., MCIHTuk

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Abstract

Natural disasters can cause severe damage to infrastructure such as the road network. Currently, a Pavement Management System (PMS) does not incorporate flooding in a Life Cycle Analysis (LCA). A few Road Deterioration (RD) models have addressed flooding, but they have limitations. As a result, there are not any comprehensive RD models that can incorporate flooding in pavement performances. In addition, no optimum life-cycle road maintenance strategy is available. No study investigated the pavement performances because of a loss in Modulus of Resilience (Mr) at granular and subgrade layers during extreme moisture intrusion. The derivation of pre- and post-flood road maintenance strategies and a flood risk assessment should also be incorporated in a PMS.

As a case study, this research has considered the January 2011 flood of Queensland, Australia, and has used 34,000 km road database of the Queensland's main roads authority. The major objectives of this study are to derive: i) network and project level roughness and rutting-based RD models with flooding, ii) pavement performances due to Mr loss at granular and subgrade layers, iii) flood-resilient pavements, iv) optimum road maintenance strategies at without flood, pre- and post-flood scenarios, and v) pavements' flood risks. The current scope covered the pavements that are affected after a flood, but are not washed away completely and need rehabilitation for structural strengthening.

This research has used a probabilistic approach for deriving the RD models, which are valid at both network and project levels. Moreover, the proposed RD models can estimate road deterioration after a flood at different probabilities of flooding.

The actual roughness and rutting vs. time data are assessed for the representative road groups or site specific roads to get the transition probability matrices for with and without flooding conditions, which are used in a Monte Carlo simulation. The new RD models show significant pavement deterioration at different probabilities of flooding events. The results are found valid with actual data for about 2 to 3 years after the January 2011 flood. A t-test also supports this match. A pavement's performance due to Mr losses at granular layers is checked using the two renowned roughness models, which results are found close match with the actual after flood data and RD models.

All these results are used estimating pavements flood resilience from three techniques, i.e., i) using the RD modelling results and an indicator of Change in International Roughness Index (ΔIRI) in year 1 over the probability of flooding (ΔIRI/Pr); ii) with the indicator of ΔIRI in year 1 over Mr loss (ΔIRI/MrL); and iii) using the flood-risk consequences. Expectedly, a flood-resilient pavement performed better in the life-cycle.

The study has derived optimum pavement maintenance strategies at without flood, pre- and post-flood conditions for the Queensland roads authority as a case-study. About $17.8bn is needed in the next 20 years at normal condition to maintain its flexible and composite roads at 4.0 IRI. The post-flood strategy framework uses the new RD models for predicting after flood deterioration and the Highway Development and Management model (HDM-4) for getting optimum solutions. The unconstrained budget solution requires $49.7bn to keep the network at an excellent condition, while the constrained one provided a reasonable solution with about $26.1bn in life-cycle. The pre-flood maintenance strategy considered an effective approach by upgrading a pavement's structural strength now with a thin overlay, and then evaluating pavement life-cycle performance if a flood comes in different years for predicting after-flood deterioration using the RD models and selecting cost effective treatments utilising the HDM-4. The total pre-flood strategy cost varied within the range of $37bn to $38bn. Comparing to a post-flood strategy, the pre-flood strategy can maintain the network better and provides positive economic benefits.

Finally, the current study aimed to evaluate pavement performances before and after a flood. The roughness vs. time data were used to get a flood and the time gap between two floods was considered as likelihood. Distribution changes of roughness data before and after a flood have been used to calculate flood consequences, and then risk results. The flood consequence and risk results were validated with actual data for two road groups.

A road authority can use the RD models for an after flood road deterioration prediction to select appropriate post-flood treatments. Moreover, pavements flood-resilience, life-cycle performances and approaches for the pre- and post-flood strategies may be used for efficiently managing roads with flooding. Any road authority may plan converting flood prone roads into resilience ones for better life-cycle performances. The pavement's flood resilience has addressed one of the major challenges of climate change and could be used for an innovative pavement design. The research findings are useful for sound strategic planning and sustainable road asset management, as it addresses the impact of flooding events in LCA. All these help in improving a PMS incorporating flooding.

Declaration by author

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Publications during candidature

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1. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2014). **Development of Road Deterioration Models Incorporating Flooding for Optimum Maintenance and Rehabilitation Strategies.** Road and Transport Research, ARRB, Australia, Volume 23, No. 1, pp 3-24. (http://www.arrb.com.au/Information-services/Publications/Issue.aspx?id=50).
2. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2014). **Developing a New Road Deterioration Model Incorporating Flooding.** Proceedings of the Transport, ICE, UK, Volume 167, Issue 5, pp 322-333. (http://www.icevirtuallibrary.com/content/article/10.1680/tran.13.00095).
3. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2015). **Development of a Post-flood Road Maintenance Strategy: Case Study Queensland, Australia.** International Journal of Pavement Engineering, UK, DOI: 10.1080/10298436.2015.1121781. (http://dx.doi.org/10.1080/10298436.2015.1121781).
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5. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2017). **Assessment of Flood Risks to Performance of Highway Pavements.** Proceedings of the Transport, ICE, UK, Volume 170, Issue 1, pp 1-10. (http://www.icevirtuallibrary.com/doi/abs/10.1680/jtran.15.00120).
6. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2017). **Estimating Pavement's Flood Resilience.** Journal of Transportation Engineering, Part B: Pavements, ASCE, Volume 143, Issue 3. (http://ascelibrary.org/doi/abs/10.1061/JPEODX.0000007).
7. Khan, M.U., M. Mesbah, L. Ferreira and D. Williams (2017). **Development of Optimum Pavement Maintenance Strategies for a Road Network.** Paper accepted for publication to the Australian Journal of Civil Engineering, Australia. (http://www.tandfonline.com/doi/full/10.1080/14488353.2017.1362823).

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Keywords

Pavement management, flooding, road deterioration models with flooding, flood resilience, pavement performance, optimum maintenance strategy, post-flood strategy, pre-flood strategy, flood risks assessment

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Fields of Research (FoR) Classification

FoR code: 0905, Civil Engineering, 100%

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List of abbreviations

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highways and Transportation Official
AASHO	American Association of State Highway Officials
AC	Asphalt Concrete
ANN	Artificial Neural Network
ARRB	The Australian Road Research Board
ARRB TR	The Australian Road Research Board model
BoM	Bureau of Meteorology
BRIDIT	Markov chain based bridge management software
CBA	Cost Benefit Analysis
CBR	Cost Benefit Ratio
C_HT_F	Composite pavement with High Traffic loading and Fair strength road group
C_HT_P	Composite pavement with High Traffic loading and Poor strength road group
C_HT_S	Composite pavement with High Traffic loading and Strong strength road group
C_LT_F	Composite pavement with Low Traffic loading and Fair strength road group
C_LT_P	Composite pavement with Low Traffic loading and Poor strength road group
C_LT_S	Composite pavement with Low Traffic loading and Strong strength road group
C_MT_F	Composite pavement with Medium Traffic loading and Fair strength road group
C_MT_P	Composite pavement with Medium Traffic loading and Poor strength road group
C_MT_S	Composite pavement with Medium Traffic loading and Strong strength road group
CRC CI	Cooperative Research Centre for Construction Innovation
CV	Commercial Vehicle
Df	Degrees of Freedom
DP	Dynamic Programming
dTIMS	Deighton's Total Infrastructure Management System

EICM	Enhanced Integrated Climatic Model
ESAL	Equivalent Single Axle Loading
FEM	Finite Element Method
F_HT_F	Flexible pavement with High Traffic loading and Fair strength road group
F_HT_P	Flexible pavement with High Traffic loading and Poor strength road group
F_HT_S	Flexible pavement with High Traffic loading and Strong strength road group
F_LT_F	Flexible pavement with Low Traffic loading and Fair strength road group
F_LT_P	Flexible pavement with Low Traffic loading and Poor strength road group
F_LT_S	Flexible pavement with Low Traffic loading and Strong strength road group
F_MT_F	Flexible pavement with Medium Traffic loading and Fair strength road group
F_MT_P	Flexible pavement with Medium Traffic loading and Poor strength road group
F_MT_S	Flexible pavement with Medium Traffic loading and Strong strength road group
FNOS	Financial Planning and Network Optimization System
GA	Genetic Algorithm
GDP	Gross Domestic Product
GP	Goal Programming
HDM-III	Highway Design and Maintenance Standards model
HDM-4	Highway Development and Management model
HV	Heavy Vehicle
IP	Integer Programming
IRI	International Roughness Index
IRR	Internal Rate of Return
K-M	Kaplan-Meier method
LoS	Level of Service
LP	Linear Programming
LCA	Life Cycle Analysis
MCA	Multi Criteria Analysis
MCMC	Monte Carlo Markov Chain
MESAL	Million Equivalent Single Axle Loading
MET	Method of Equivalent Thickness
MPa	Mega Pascal

M&R	Maintenance and Rehabilitation strategies
MT	Motorized Vehicle
Mr	Modulus of Resilience
NLP	Non-linear Programming
NPV	Net Present Value
NMT	Non-Motorized Vehicle
OV	Overlay
PONTIS	Markov model for bridge management system, which was used by the Arizona Dept. of Transport
PM	Periodic Maintenance
PMS	Pavement Management System
RD	Road Deterioration models
RIMES	The Road Infrastructure Maintenance Evaluation Study
RISKO	Risk assessment model being used by TMR-QLD
RM	Routine Maintenance
R_HT_F	Rigid pavement with High Traffic loading and Fair strength road group
R_HT_P	Rigid pavement with High Traffic loading and Poor strength road group
R_HT_S	Rigid pavement with High Traffic loading and Strong strength road group
R_LT_F	Rigid pavement with Low Traffic loading and Fair strength road group
R_LT_P	Rigid pavement with Low Traffic loading and Poor strength road group
R_LT_S	Rigid pavement with Low Traffic loading and Strong strength road group
R_MT_F	Rigid pavement with Medium Traffic loading and Fair strength road group
R_MT_P	Rigid pavement with Medium Traffic loading and Poor strength road group
R_MT_S	Rigid pavement with Medium Traffic loading and Strong strength road group
RMS, NSW	The Road and Maritime Services, New South Wales
S	Moisture Content
S_{opt}	Optimum Moisture Content
SCENARIO	Decision-making model being used by TMR-QLD
SN	Structural Number
TMR-QLD	The Transport and Main Roads, Queensland
TNRP	The Transport Network Reconstruction Program of TMR-QLD
TPM	Transition Probability Matrix
TOBJ	Total Objective Function
T1_F2_RD3&4_Post3	Treatments in year 1, a flood in year 2, RD models used in year 3 & 4 and

	post flood treatments start from year 3
T1_F4_RD5&6_Post5	Treatments in year 1, a flood in year 4, RD models used in year 5 & 6 and post flood treatments start from year 5
T1_F7_RD8&9_Post8	Treatments in year 1, a flood in year 7, RD models used in year 8 & 9 and post flood treatments start from year 8
T1_F12_RD13&14_Post13	Treatments in year 1, a flood in year 12, RD models used in year 13 & 14 and post flood treatments start from year 13
T1_F17_RD18_Post18	Treatments in year 1, a flood in year 17, RD models used in year 18 and post flood treatments start from year 18
T0_F2_RD3&4_Post3	No treatments in year 1, a flood in year 2, RD models used in year 3 & 4 and post flood treatments start from year 3
T0_F4_RD5&6_Post5	No treatments in year 1, a flood in year 4, RD models used in year 5 & 6 and post flood treatments start from year 5
T0_F7_RD8&9_Post8	No treatments in year 1, a flood in year 7, RD models used in year 8 & 9 and post flood treatments start from year 8
T0_F12_RD13&14_Post13	No treatments in year 1, a flood in year 12, RD models used in year 13 & 14 and post flood treatments start from year 13
T0_F17_RD18_Post18	No treatments in year 1, a flood in year 17, RD models used in year 18 and post flood treatments start from year 18
VOC	Vehicle Operating Costs
WE	Work Effects models
WNR	Wet Non-reactive
ΔIRI	Incremental Change in Roughness
$\Delta IRI_{0\%}$	Incremental change in roughness at time t due to a 0% probability of flooding (i.e. at normal condition)
$\Delta IRI_{20\%}$	Incremental change in roughness at time t due to a 20% probability of flooding
$\Delta IRI_{f20\%}$	Incremental change in roughness at time t due to a 20% probability of flooding only
ΔRutting	Incremental Change in Rutting
$\Delta Rutting_{0\%}$	Incremental change in rutting at time t due to a 0% probability of flooding (i.e. at normal condition)
$\Delta Rutting_{20\%}$	Incremental change in rutting at time t due to a 20% probability of flooding
$\Delta Rutting_{f20\%}$	Incremental change in rutting at time t due to a 20% probability of flooding only

ΔIRI/Pr	ΔIRI in year 1 over probability of flooding
ΔIRI/MrL	ΔIRI in year 1 over Mr loss
ε_c	Compressive Strain at the Top of Subgrade
ε_t	Tensile Strain at the Bottom of Bituminous layer

The Thesis is Dedicated to my

Parents, Wife and Daughters

CHAPTER ONE: INTRODUCTION

1.1 Background

Natural disasters can cause severe damages to infrastructure, including roads. Recent relevant serious natural disasters worldwide include the 2012 tsunami in Japan, 2011 earthquakes in Haiti and New Zealand, hurricanes Katrina in 2005 and Sandy in 2012 in the USA and 2010-11 and 2013 flooding in Australia. From a pavement management perspective, two conditions are expected for a road segment after a natural disaster such as flooding. The road may be substantially damaged to an extent that it should be reconstructed. Alternatively, the road may be serviceable after the disaster. In the latter case, the road structure may remain inundated for several days which lead to a weakened base and subbase. In all these cases, the road assets would be seriously damaged, indicating that a better Pavement Management System (PMS) incorporating natural disasters is urgently needed for an efficient preservation of the road system.

Studies on pavement responses due to flooding are limited. Helali, et al. (2008) covered the impact of hurricanes Katrina and Rita, occurred in 2005 in Louisiana in the USA, on pavement performances. Roads were submerged for weeks and traffics with heavy loading were moving over these flooded roads. It was found that 90 to 190 mm of Asphalt Concrete (AC) overlay were required as rehabilitation for the flooded roads to enhance structural strengths. They also found that the flooded sections deteriorated more than the controlled sections with 2.5 to 6.5 times higher deflection values. Another study revealed that the average pavement strength and subgrade modulus loss were 18% and 25% respectively due to flooding. The flooded pavements had higher deflection, and as a result they had lower strength and modulus (Zhang, et al. 2008). A flood reduces moduli of pavement layers, and as a consequence a pavement’s strength decreases and deflection increases. These indicate poor pavement responses to flooding.

The 2010/11 flooding in Queensland of Australia was the worst for 30 years, which affected 70% of the state equal to an area of France and Germany combined and 60% of the state’s population. The indicative loss to the economy was about A$13bn to A$30bn (1-2.5% of Gross Domestic Product, GDP), and road asset loss was around A$2.8bn and total damage to public infrastructure was between A$5bn to A$6bn. In addition to the flood, cyclone Yasi added another A$800m loss to the road and transport network (PWC, 2011). A recent assessment revealed that about 28% of major roads (9,170 km) were severely damaged during these events and 300 roads together with nine major highways were closed (TMR, 2012a). The June 2012 progress report reveals that the Department of Transport and Main Roads, Queensland (TMR-QLD) could not manage to complete its largest program (‘Operation Queenslanders’ of $4.2bn) to reconstruct 6,709 km of roads (TMR, 2012b). In general, Australia is affected by flooding due to low-lying areas in some places. Intensity of flooding and water ponding varies from storm to storm. The frequency of flooding is also uncertain, for example, Queensland experienced flooding in 2010/11 and again in 2013.

All these indicate that a natural disaster such as flooding effects on the pavement performances. As a result, appropriate deterioration prediction after a flood and post-flood treatments selection are vital for managing flood damaged roads. The current research has focused on investigating this problem for improving a PMS with a flood.

1.2 Research Aims and Objectives

Every road authority should have a comprehensive PMS to manage its road assets efficiently (Haas, et al. 1994). A PMS optimises overall performance of a road network within the allocated resources (OECD, 1994; Robinson, et al. 1998). Generally, it consists of: i) data collection, ii) a good quality database, iii) a decision-making tool incorporating Road Deterioration (RD) models, Work Effects (WE) models, engineering judgment on criteria and optimisation, iv) implementation procedures, and v) feedback (Battiato, et al. 1994; Mills, 2010).

It is certain that a PMS will be improved if a flooding is considered in pavement Life Cycle Analysis (LCA). Development of RD models with flooding provides pavement performances after a flood. It helps selecting appropriate treatments and a road maintenance strategy. The current research has aimed to address these by deriving the followings:

1.2.1 Development of new roughness and rutting based RD models with flooding at project and network levels

This is the major objective of the research. It aims to develop novel site-specific and network level roughness and rutting based RD models with flooding. The models are valid for short period up to a certain time after the flood. The models are verified with actual data.

1.2.2 Assessing pavement performance due to loss of modulus at granular layers

Pavement performances are assessed due to granular and subgrade layers' Modulus of Resilience (Mr) losses during flooding. It provides a verification of the roughness based RD models.

Generally, flooding, heavy rainfall with water ponding, localised flooding and poor sub-surface drainage may increase moisture in pavement layers. In the current study, this extreme moisture intrusion may be termed as a flood. This helps understanding a road's performance since loss of Mr due to moisture intrusion is a typical scenario for flooding.

1.2.3 Obtaining flood-resilient pavements

Another basic work is to use the two new indicators for getting flood-resilient pavements. The new RD models with flooding and their verification results are used for this purpose. It gives reliable pavement performance results after flooding.

1.2.4 Development of optimum, pre- and post-flood road maintenance strategies

A road authority requires maintenance strategy at normal condition and also including flooding when RD models are developed. Therefore, optimum, pre- and post-flood strategies are derived for a road authority.

It is observed that the TMR-QLD does not have set maintenance standards to trigger treatments, thus, its investments may not be cost-effective (TMR, 2002). Therefore, an optimum maintenance strategy at without flood conditions should be first to be developed. The new RD models support developing pre- and post-flood road maintenance strategies.

1.2.5 Flood risk assessment

Impact of a flood on pavement performances has been evaluated, which gives a picture of flooding consequences and risks. The flood risk assessment is done for the network level.

The literature review related to PMS and the impact of flooding on pavement performances helps identifying all the above knowledge gaps. After a detailed literature review, it is suggested that a probabilistic RD model incorporating uncertainty like flooding to be used for asset management. Moreover, derivation of a pre- and post-flood strategy incorporating flooding with pavement' LCA is necessary. The loss of Mr at granular and subgrade layers due to moisture intrusion/flooding and its effect would provide useful feedback for efficient road preservation. Furthermore, it has aimed to analyse the flood risks for different road groups of the network based on risk scoring and distribution; which would help to plan for a better road network.

The scope of this research covers the flood damaged pavements, which were saturated but the embankment and structure have remained intact (not completely damaged or washed away), that are at moderate risk of further flooding and need preventive maintenance and rehabilitation with or without partial reconstruction. These roads need appropriate attention before and after a flooding event. If a road segment is totally damaged, the treatment is a simple (but costly) treatment of reconstruction. Since, there is only one option available, no optimisation is required for such segments.

As a particular case study, the 2010-11 flooding in Queensland, Australia has been considered. Queensland is particularly prone to flooding from cyclonic events, and experienced a similar magnitude of flooding in early 2013. Therefore, 34,000 km road database of the TMR-QLD has been used in the study. It is worth mentioning that this database has after-flood road condition data captured.

1.3 Thesis Significance and Contribution

Currently, a PMS does not include flooding. For example, TMR-QLD does not have appropriate deterioration prediction models with flooding to select optimum treatments, and after flood investigation of pavements is not detailed. As a result, the research targets to improve road asset management practices incorporating flooding. It ensures an appropriate road deterioration prediction at project and network levels after a flood; and helps providing adequate treatments with optimum budget. As a result, it assures a better value for money.

The outcomes will make sure the maintenance philosophy of providing the right treatment, at the right time and right place. The pre- and post-flood road maintenance strategies provide sound approaches to tackle flooding in pavements' life-cycle. Hence, they ensure managing roads at set targets/standards with optimum budget. The optimum strategy at normal conditions assists managing assets efficiently.

The results on pavements flood resilience and flood risk assessment also provide pavement performances after a flood. The flood resistance and risks/consequences observed from pavement performances after flooding can be used as indicators in pavement design. In addition, it reveals that the identified vulnerable pavements may be managed before a flood comes.

All these will certainly improve a PMS through the use of new roughness and rutting based RD models, appropriate treatment selection and optimum pre- and post-flood strategies. Moreover, the results obtained from the flood resilient pavement analysis and risk assessment help converting flood damaged pavements into sound ones for better life-cycle performances with flooding.

1.4 Thesis Outline

Figure 1.1 shows the thesis outline. The thesis has eleven chapters covering introduction, literature review, methodology, analysis, results and conclusions. Chapter One focuses on background, objective, scope and importance of the study.

Chapter Two provides a detailed literature review to develop new roughness and rutting-based RD models; pre- and post-flood and optimum Maintenance and Rehabilitation (M&R) strategy for a road network to appropriately address flooding; to assess impact of the Mr loss on road performances and risk analysis. It highlights the existing PMS in Australia; the importance of RD models (deterministic and probabilistic) in a PMS and the use of probabilistic models for road deterioration prediction. The study has found that a probabilistic RD model can better address the uncertainty of pavement behaviour. Hence, details of Markov Chain, homogenous and non-homogeneous Transition Probability Matrices (TPM) have been studied. The chapter identifies six methods for deriving RD models using a non-homogeneous TPM, i.e., minimum error method, percentage transition method, probit model, conversion from deterministic model, Bayesian technique and multi-stage hazard model. Roughness progression prediction has been done with two renowned models, i.e., American Association of State Highways and Transportation Officials (AASHTO) and Highway Development and Management model (HDM-4), due to moister intrusion and consequent Mr losses at granular and subgrade layers, which are discussed in details. It shows the optimisation algorithm to obtain pre- and post-flood road maintenance strategies and M&R

strategy at without flood condition. Relevant literatures on risk assessment are also highlighted in this chapter.

Chapter Three includes the detailed methodology in deriving the roughness and rutting-based RD models at project and network levels, assessing impact of Mr losses at granular and subgrade layers on pavement performances, flood-resilient pavements, pre- and post-flood strategies, optimum M&R strategy, and flood risk assessment. The proposed RD models are based on non-homogeneous TPMs, as they can consider abrupt changes in pavement deterioration due to flooding. As Mr of granular and subgrade materials are sensitive to moisture content, therefore, Mr loss due to flooding has a direct influence on pavement performances, which was supported with previous flooding data obtained from the hurricane Katrina. In view of that a sensitivity analysis of Mr loss on pavement performances has been done. Both the pre- and post-flood road maintenance strategies are derived using an Integer Program with optimisation objective of 'minimise agency cost and maximise performance target at budget constraint'. The HDM-4 model has been utilised for a life-cycle cost-benefit analysis to obtain these strategies for the whole road network. It has also been used for deriving optimum maintenance standards and strategies for the TMR-QLD. The study has aimed to develop a risk assessment framework incorporating flooding.

Chapters Four to Ten provide all the major findings. The new roughness and rutting-based RD models for two road groups are shown as an example. These RD models can predict road deterioration at different probabilities of flooding up to 2 to 3 years after a flooding event. These are derived using non-homogeneous TPMs and Monte Carlo simulation. The project and network levels RD models and their validation with actual data are shown.

Pavement performances with flooding due to Mr loss are shown for seven flexible pavement road groups, which provided reliable comparison results with the actual data and new RD models. As a result, the roughness prediction after a flood using the AASHTO and HDM-4 also verified the RD models.

Flood resilient pavements are obtained using the RD modelling results and findings derived from the Mr loss analysis. The study has proposed two new gradients: i) gradient of Incremental Change in International Roughness Index (ΔIRI) in year 1 over probability of flooding (ΔIRI/Pr); and ii) ΔIRI in year 1 over Mr loss (ΔIRI/MrL) using the ΔIRI vs. % probability of flooding and ΔIRI vs. % Mr loss relationships, respectively. These gradients can help quantify flood-resilient pavements.

The HDM-4 optimisation analyses produce the new pre- and post-flood road maintenance strategies as well as the M&R strategy at normal condition for the TMR-QLD. Finally, a flood risk assessment has been shown for different road groups of Queensland.

The Chapter Eleven closes the report with conclusions. It shows major findings on the new RD models with flooding; optimum, pre- and post-flood strategies; flood resilience and flood risks of pavements. The study limitations and future work plans are proposed.

It is demonstrated that the new RD models will help better road deterioration prediction with flooding. It will help in sound decision-making and appropriate treatments selection. The derived flood resilient pavements give suggestion for converting the existing flood damaged prone roads to better resistant one. This helps in long-term pavement performances with flooding. The new pre- and post-flood strategies assist managing a road network efficiently with flooding and propose requesting for necessary budget. In addition, the flood risk and consequence scores support a road authority to take an appropriate attention on the vulnerable roads. All these help improving a PMS incorporating flooding.

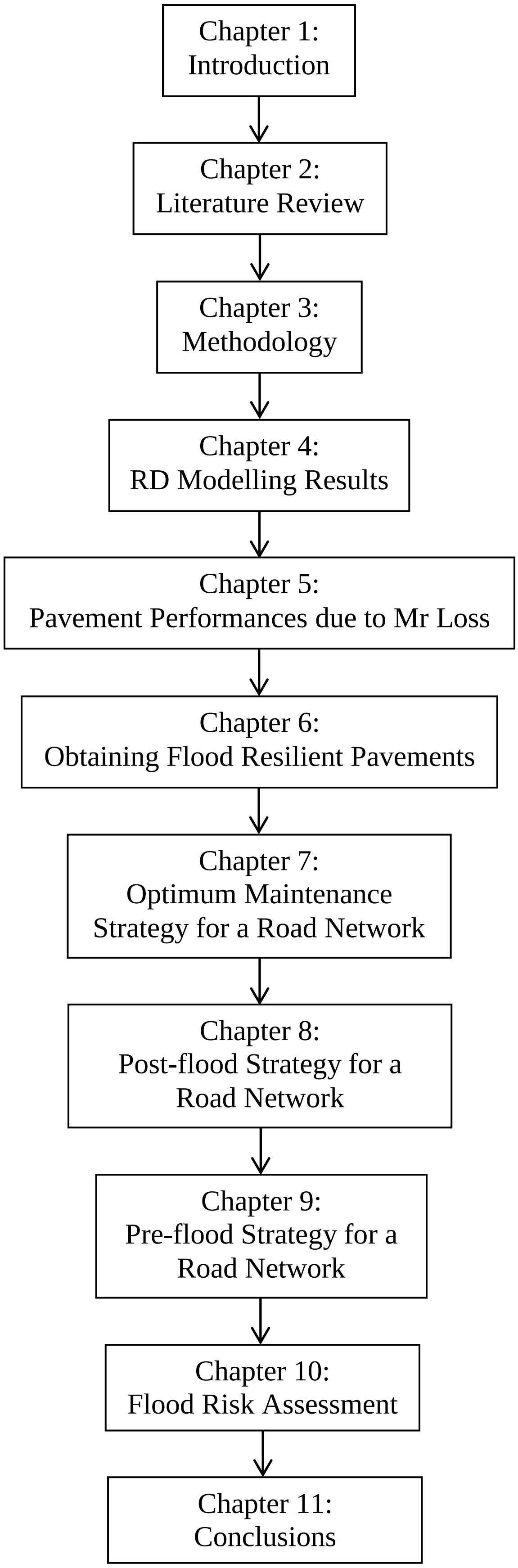

Figure 1.1: Thesis outline

CHAPTER TWO: LITERATURE REVIEW[1]

A detailed literature review is presented in this chapter. *Figure 2.1* shows a broad area of a PMS covered in the current literature review, i.e., RD models and types, WE models, optimisation for obtaining an M&R strategy, the effect of flooding on pavement performances and risk assessment.

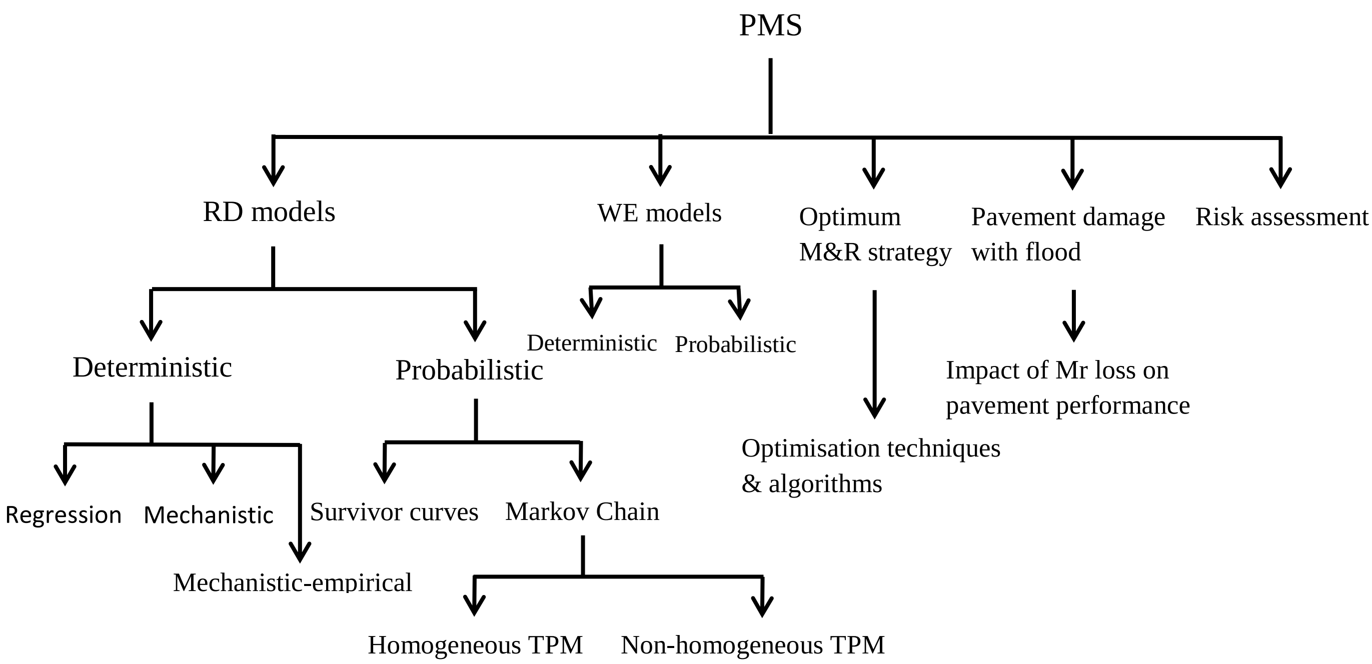

Figure 2.1: Broad areas considered in the literature review

2.1 PMS in Australia

The Australian Road Research Board (ARRB) discussed about a PMS in the early 1970s (Haas and Hutchinson, 1970). Austroads (2009a) stated that a PMS may include stakeholder/community expectations; formulation and review of asset strategies; development of an investment program (level of service standards, road hierarchy, target and current levels of service and asset performance gap analysis); identification of assets requirements (total needs program, optimisation and/or prioritisation and funding scenarios); implementation of a work program; audit of works carried out and review of completed works. The Austroads' asset management guidelines discussed the best road management practices for better performances and benefits to all road transport stakeholders.

[1] *This chapter has resulted a Journal Paper: http://www.arrb.com.au/Information-services/Publications/Issue.aspx?id=50*

Almost each road authority in the world has a PMS in place. This Chapter has mainly highlighted the PMS status in the major road authorities of Australia because of its scope. The main roads departments in different states of Australia have established PMSs for road preservation appropriately (see *Table 2.1*). All road authorities use a PMS as a tool for Cost Benefit Analysis (CBA), optimisation and prioritisation for efficient asset management.

As mentioned, the current study has intended to address the PMS in Queensland as a case study. The TMR-QLD has a Road Asset Management unit responsible for managing about 34,000 km of roads. Its PMS consists of road condition, traffic and roughness data collection; a database; decision making model (SCENARIO); risk assessment model (RISKO) after obtaining results from the SCENARIO; programming; and budgeting. The authority uses roughness, rutting and cracking deterioration models in SCENARIO. The TMR-QLD's PMS was established in the early 90s and the RD model was developed for local use (TMR, 2002).

Table 2.1: Comparison of PMS among different main roads departments in Australia

Department	Road Length (km)	Asset Value (A$bn)	PMS Exists	RD Models	Decision-Making Tools	Optimisation and Prioritisation	Data Source
Transport and Main Roads, Queensland (TMR-QLD)	34,000	23	Yes; started 1990s	Roughness: ARRB TR Rutting: regression based Cracking: HDM-III	SCENARIO (CBA)	Maximises roughness or road agency budget constraint. Multi Criteria Analysis (MCA) for prioritisation (uses RISKO model).	(TMR, 2002)
Main Roads, Western Australia	18,000	27	Yes	-	Uses CBA	Maximises benefits. Uses MCA (CBA, social and environmental).	(Li and Kumar, 2003a; Main Roads, 2014)
Road and Maritime Services, New South Wales (RMS, NSW)	40,000	39	Yes; started late 1980s	Used earlier probabilistic road deterioration models	Uses CBA. Used earlier FNOS based on probabilistic RD models.	Uses Cost Benefit Ratio (CBR) for prioritisation.	(Li and Kumar, 2003a; Li and Kumar, 2003b; Porter, 1987)
Main Roads, Victoria (VicRoads)	23,000	-	Yes; started 1990s	-	HDM-4 (CBA)	Minimises life-cycle costs. Analytical Hierarchy Process for MCA. Geographical Information System.	(Anderson, et al. 1994; Li and Kumar, 2003a)
Dept. of Transport, Energy and Infrastructure, South Australia	23,000	6	Yes; started early-mid 1990s	-	Uses CBA (Deighton's Total Infrastructure Management System, dTIMS software)	Maximises best maintenance at budget constraint. (uses dTIMS software).	(Dept. of Transport, 2012; Li and Kumar, 2003a; Mundy, 2012)

2.2 RD Models

A RD model shows pavement deterioration/performance trend in its life-cycle. To obtain a sound network level maintenance program within a stipulated budget, a reliable RD model is necessary to assess pavement performances (Haas, et al. 1994). Optimum M&R strategies are also based on road deterioration trends. Therefore, RD models are the key component of a PMS.

Generally, there are two types of RD models, namely deterministic and probabilistic models. The deterministic models provide specific values on deterioration; whereas, the probabilistic approach relies on probable changes in condition states. Several studies have mentioned the importance of pavement deterioration prediction, and have discussed deterministic and probabilistic models for pavement performances assessment (Chun, et al. 2012; Li, et al. 1995; Li, et al. 1996; Li, 2005; Madanat, et al. 1995; Martin, 2009; Prozzi, 2001; Ranjith, et al. 2011; Robinson, et al. 1998; Tack and Chou, 2002). Moreover, Austroads (2009b) and Panthi (2009) elaborated the RD modelling methods, and identified assumptions and limitations for different pavement performance models.

2.2.1 Deterministic models

Deterministic models are a type of mathematical model in which outcomes are precisely determined through known relationships among states and events. Li, et al. (1995 and 1996) and Robinson, et al. (1998) have discussed different types of deterministic models; which include regression, mechanistic and mechanistic-empirical models.

Regression analysis is a statistical approach based on historical data, and it uses one or more independent variables to obtain a dependent variable. Regression modelling is easy to develop and needs less time and data storage. However, it needs a large database for a better model which can work only within the range of input data. In addition, sometimes faulty data induces poor prediction, and the selection of the correct data is difficult and time consuming (FHWA, 1990). Hence, Madanat, et al. (1995) concluded that statistical regression is not a suitable method to assess pavement uncertainty.

Mechanistic models predict cause and effect to give the best results which are primarily based on theory, i.e., stress, strain and deflection. This type of model uses large numbers of variables, for example, material properties, environmental conditions, geometric elements, loading characteristics, etc. and can only predict basic material responses (FHWA, 1990). Data for these variables are difficult to obtain (Gucbilmez and Yuce, 1995). Both Panthi (2009) and Gucbilmez and Yuce (1995) have found that mechanistic models are complex in nature and relate to pavement design parameters which do not show real pavement performances.

Similar to the mechanistic model, a mechanistic-empirical model is also based on a cause and effect relationship but its prediction are better. It is easy to work with a final empirical model because it needs less computer power and time. However, the method depends on field data for the development of an empirical model and works within a fixed domain of independent variables. In addition, it works with a large number of variables (e.g., material properties, environmental conditions, geometric elements, loading characteristics, etc.) which are often not available in a PMS (FHWA, 1990; Gucbilmez and Yuce, 1995); and therefore, it is a complex model (Gucbilmez and Yuce, 1995; Panthi, 2009).

The well-known PMS tools (Highway Design and Maintenance Standards model (HDM-III); HDM-4 and Australian Road Research Board, ARRB TR models, etc.) are mechanistic-empirical models. The TMR-QLD uses the SCENARIO which is based on deterministic models (TMR, 2002), that comprises the roughness model established on the ARRB TR model (mechanistic-empirical); the cracking model based on HDM-III (mechanistic-empirical); and the rutting model built on linear extrapolation.

2.2.2 Probabilistic models

The probabilistic models predict road deterioration to capture the uncertainty of traffic and environmental variables. Ranjith, et al. (2011) used Australian data from the VicRoads and two regional councils of Victoria, to develop probabilistic deterioration models for timber bridges. Li (2005) suggested to consider stochastic and dynamic modelling for pavement deterioration predictions. Several studies have highlighted the importance of the probabilistic models as they are useful in dealing with uncertainty related to a pavement's behaviour in its design phase (Li, 1997; Li, et al. 1995; Li, et al. 1996; Madanat, et al. 1995; Mandiartha, 2010; Ortiz-Garcia, et al. 2006; Tack and Chou, 2002). Pavement materials have indeterminate characteristics due to variable traffic loading, temperature and moisture. Therefore, a pavement's performance is not always appropriately anticipated.

Generally, probabilistic models are of two types, that is, the Markov process and survivor curves (Mills, 2010). A survivor curve is easy to derive, but it only provides a probability of failure vs. age relationship. Moreover, considerable error may be expected if a small group of units are used (FHWA, 1990). This curve is not related to design variables, rather only links to the pavement age (Panthi, 2009).

The simple Markovian approach is a well-established way to reflect pavement performance trends appropriately. However, it does not provide guidance on physical factors which contribute to change and it does not consider past performance (FHWA, 1990; Ortiz-Garcia, et al. 2006). On the other hand, the Semi-Markov method considers past performance, which can be developed solely on subjective inputs with less field data. Similar to Markov, this method provides a way to incorporate data feedback (FHWA, 1990).

In general, the deterministic models provide an outcome using physical and functional factors of a pavement, but they are difficult to develop and relate to basic pavement responses. On the other hand, the probabilistic models are more realistic and obtain actual pavement performances. Pavement deterioration is dependent on several key factors, that is, material characteristics, loading and environment; all of them are stochastic in nature. Therefore, it is sensible to use the probabilistic models (Chun, et al. 2012; Li, 1997), as they can consider uncertainty of pavement performances due to traffic and environmental variables. The following sections discuss detailed of a Markov Chain.

2.2.3 Markov Chain review

A Markov chain is a mathematical system that undergoes transitions from one state to another, where the next state depends only on the current state and not on the sequence of events that preceded it. The Markov chain has been used for the infrastructure deterioration predictions, including bridges (Fu and Debraj, 2008; Mishalani and Madanat, 2002; Ranjith, et al. 2011), roads (Abaza and Ashur, 1999; Anderson, et al. 1994; Butt, et al. 1994; Hudson, et al. 1998; Jin, et al. 2010; Kobayashi, et al. 2010; Lethanh and Adey, 2012; Li, 1997; Li and Haas, 1998; Madanat, et al. 1995; Mandiartha, 2010; Mills, 2010; Ortiz-Garcia, et al. 2006; Panthi, 2009; Pierce, 2003; Robinson, et al. 1998; Tack and Chou, 2002; Wang, et al. 1994), waste water (Hyeon-shik, et al. 2006), rail (Ferreira and Murray, 1997; Shafahi and Hakhamaneshi, 2009) and pipelines (Jin, et al. 2010; Sinha and Mark, 2004).

One of the major advantages of using the Markov model is that it has the capacity to integrate pavement deterioration rates and M&R improvement variables into a single entity which is the transition matrix. As a result, accurate pavement deterioration predictions using stochastic and dynamic load modelling and an optimal maintenance strategy can be easily derived (Butt, et al. 1994; Chun, et al. 2012; Li, 1997; Li, et al. 1996; Li, 2005).

The Arizona Department of Transport in America has been using the Markov model in their PONTIS bridge management systems (Mandiartha, 2010; Wang, et al. 1994). PONTIS uses a homogeneous Markov chain (Fu and Debraj, 2008). Another bridge management software, BRIDIT, is also based on the Markov chain (Panthi, 2009). In Australia, the Road and Maritime Services, New South Wales (RMS, NSW) developed the Financial Planning and Network Optimisation System (FNOS) based on the Markov chain (Mandiartha, 2010).

The Markov chain may or may not be time-dependent. Time independent Markov chains are known as homogeneous TPM. However, it may not be realistic as a pavement's deterioration and behaviour varies with time. Therefore, homogeneous deterioration predictions do not fit with real conditions since they assume the probability matrices as constant (Butt, et al. 1994; Fu and Debraj, 2008; Li, et al. 1996). When a transition probability is assumed to change with time, it is known as a non-homogenous (Semi-Markov) Markov chain; and its' TPM is known as a non-homogeneous TPM.

Numerous studies have used the Markov chain for RD modelling purposes using homogeneous and non-homogeneous TPMs. Mbwana and Turnquist (1996) and Kuhn and Madanat (2005) used Markov transition probability. Chun, et al. (2012) utilised the Kaplan-Meier (K-M) method to derive cumulative transition probability. Li, et al. (1996) considered both the homogeneous TPM with the Chapman-Kolmogorov equation and non-homogeneous TPM in their analysis. Wang, et al. (1994) used the Chapman-Kolmogorov equation for n-step transition matrices. Robinson, et al. (1998) recommended a homogenous TPM. Several studies used a homogeneous TPM for RD modelling (Chun, et al. 2012; Jin, et al. 2010; Karan, 1977; Wang, et al. 1994). Similarly, various authors (Abaza, 2006; Fu and Debraj, 2008; Ortiz-Garcia, et al. 2006; Tjan and Pitaloka, 2005; Wang, et al. 1994) provided detailed descriptions and theories on developing mainly homogeneous and non-homogeneous TPMs. Tjan and Pitaloka (2005) utilised another technique, the Gaussian elimination method to derive a TPM. Shafahi and Hakhamaneshi (2009) used a 2-stage non-homogeneous TPM to study rail deterioration which meant they considered it from its current condition to its next condition only. In addition, Panthi (2009) used TPMs for RD and WE modelling.

Let: pij = probability that a road is currently in state i and will be in state j next year, and
P = [pij] is the transition matrix which shows the probabilities of various state transitions. For a 2-stage transition, the non-zero element of the matrix 'P' consists of diagonal and adjacent lower diagonal elements. In other words:

p_{ij} = 1- p_{ii} = the probability that a road currently in state i will be in state j next year, and

$\sum p_{ij} = 1$ for all i = 1, 2, ..5 and j = 1, 2, …5. (2.1)

As an example, considering 5 condition states, e.g., i = 1, 2, ..., 5 and j = 1, 2, …, 5 (1 = excellent, 2 = good, 3 = fair, 4 = poor and 5 = bad), the transition matrix becomes:

$$P = \begin{bmatrix} p11 & p12 & 0 & 0 & 0 \\ 0 & p22 & p23 & 0 & 0 \\ 0 & 0 & p33 & p34 & 0 \\ 0 & 0 & 0 & p44 & p45 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \qquad (2.2)$$

The state of a road at any time, t, t =1, 2, … can be expressed in a probabilistic manner as a (1 × 5) row vector (assuming 5 condition states) p(t):

$$p(t) = \left(p\{X(t) = 1\}, p\{X(t) = 2\}, ..., p\{X(t) = 5\}\right) \qquad (2.3)$$

Where, X(t) is the road state at time t and $p\{X(t) = i\}$ is the probability that a road is in state i at time t. Obviously, the elements of the vector p(t) must total 1.0.

Ayyub (1998) mentioned that the state vector at some future time may be calculated from the transition matrix and the initial state vector, as below:

$$\begin{aligned} &p(1) = p(0)P \\ &p(2) = p(1)P = p(0)P^2 \\ &... \\ &p(t) = p(0)P^t \end{aligned} \qquad (2.4)$$

The above *Equation 2.4* is valid for a homogeneous TPM.

2.2.4 Non-homogeneous TPM

Fu and Debraj (2008) have discussed the limitations of the PONTIS (homogeneous TPM), which are i) non-invertible matrix: as a result no TPM may be obtained; ii) negative transition probabilities which is unrealistic; and iii) possible non-zero values even though no M&R was done, which shows unrealistic change from a bad state to a good state. Therefore, they proposed to use a non-homogeneous TPM; because a homogeneous TPM is independent of time and it cannot provide the real deterioration scenario of a pavement (Butt, et al. 1994; Li, et al. 1996). Madanat, et al. (1995) used a non-homogeneous TPM for RD modelling. In another study, Mishalani and Madanat (2002) considered the Weibull hazard rate model to derive non-homogeneous transition probability, but not a TPM. Mills (2010) also considered non-homogeneous transition probability using a Monte Carlo simulation and reliability analysis.

Referring to *Equation 2.4*, the non-homogeneous state vector p(n) would be as follows:

$$
\begin{aligned}
p(1) &= p(o) * P(1)\\
p(2) &= p(1) * P(2) = p(o) * P(1) * P(2)\\
p(3) &= p(2) * P(3) = p(1) * P(2) * P(3) = p(o) * P(1) * P(2) * P(3)\\
&\ldots\ldots\\
p(t) &= p(o) * P(1) * P(2) * P(3) \ldots. * P(t) \qquad (2.5)
\end{aligned}
$$

Where, p(o) = condition vector at year zero, p(1) = condition vector at year 1, p(t) = condition vector at year t, P(1) = TPM at year 1 and P(t) = TPM at year t. However, a TPM may be derived at different time intervals, not specifically at 1-year intervals. Then, P(1) = TPM at first time interval (first step) and P(t) = TPM at t time intervals (n steps).

2.3 Recent Studies with Flooding

Sultana, et al. (2014) considered the January 2011 flooding of Queensland while developing the deterministic model of Structural Number (SN) vs. time for low volume sealed roads using deflection data. Similarly, Sultana, et al. (2016) provided a mechanistic (SN vs. time) RD model for the flexible pavements. Both the studies have some limitations. They used a sample of roads while deriving the RD models, and concentrated on traffic volume based road groups only. These studies did not consider any simulation to get the RD models for different probabilities of flooding. They were only valid for short-period, i.e., 1 to 2 months, which may not assist in treatment selection. In addition, roughness and rutting models were not derived, which helped in selecting post flood treatments. The studies did not consider developing network level RD models. Pavement performance resembled uncertainty; therefore, a probabilistic model is more justified.

The Hurricanes Katrina and Rita were also used to assess pavement performances by Chen and Zhang (2014). The study used the before and after flood roughness data for 2 years. A slightly higher roughness was found for the flooded roads. This study used road grouping based on pavement types. The real data were assessed but no RD models with flooding were generated. In addition, it did not provide details of which type of pavement performed well after the flood.

Another new study (Shamsabadi, et al. 2014) derived a simple regression model of IRI vs. time using non-continuous road data, which is valid for local conditions. The independent variables used were flooding depth, duration, loading and initial IRI, which are not always easy to collect. This model may be used for one type of road group only. In addition, a simulation based probabilistic RD model with different probabilities of flooding was not undertaken.

The above literature review reveals that many road deterioration prediction models were developed for normal condition. A few deterioration prediction models used flooding, which are regression based and deterministic models. They had several limitations as stated above. They did not consider predicting pavements deterioration for long period after a flood; however, in reality a post-flood treatment selection and its implementation need time. These studies did not consider project level or site-specific RD models with flooding. The network level road grouping was also simplistic. Moreover, no simulation was done to predict road deterioration at different probabilities of flooding.

It is found that a pavement's behaviour, a flooding event and its impact on the pavement performances are uncertain. Therefore, a probabilistic approach, especially non-homogenous Markov Chain is found suitable for the RD modelling with flooding.

Generally, a non-homogeneous TPM shows that pavement deterioration varies with time, and as a result several TPMs are required to obtain the final one. On the other hand, only one TPM is needed for a time independent analysis. Therefore, a non-homogenous TPM is recommended as it can show real pavement deterioration with time. The following section highlighted different types of methods used to derive a non-homogeneous TPM in order to finalise one for this research.

2.4 Methods of Deriving a Non-homogeneous TPM

Generally, the following six methods are used to derive a non-homogenous TPM for RD modelling. A non-homogeneous TPM is suitable as they represent real pavement performances.

2.4.1 Minimum Error method

A minimum error method describes an approach which minimises error between the observed and predicted values. Ortiz-Garcia, et al. (2006) used this method for deriving RD models with non-homogenous TPMs. They proposed three new techniques to derive a non-homogeneous TPM with minimum error objective function using i) historical data, ii) regression equation, and iii) distributions from the historical data. The analysis used six hypothetical data sets to derive TPMs. The results show that distributions from the historic data may be used to derive a TPM with the minimum error method. However, the study did not use the most representative index, that is, the International Roughness Index (IRI) (Sirirangsi, et al. 2003) for the road deterioration prediction.

Ranjith, et al. (2011) compared the minimum error and percentage transition methods for statistical analysis of timber bridge deterioration in Australia, and found the suitability of this method. They suggested a non-linear optimisation technique using the minimum error method. Similarly, Shafahi and Hakhamaneshi (2009) and Chun, et al. (2012) found good results using the minimum error method. The typical equation for the minimum error method is given below:

$$\text{Objective Function} \qquad Z = \sum_{t} C(t) - E(t)$$

$$\text{subject to} \qquad 0 \le p_{ij} \le 1 \qquad i, j = 1, 2,.., n$$
$$\sum_{j} p_{ij} = 1 \qquad i = 1, 2, \ldots, n \tag{2.6}$$

Where C (t) is the system condition rating at time t based on regression. This function describes a statistical relationship between the condition and time t, obtained by a regression analysis using the data from inspection of the pavements. E (t) is the expected rating at time t based on the Markov chain using the estimated transition probabilities.

Madanat, et al. (1995) criticised the minimum error method in deriving a TPM, as i) it does not capture various explanatory variables, for example, material properties, environmental conditions, loading scenario, etc.; ii) it considers non-homogeneity through adhoc segmentation, and iii) it cannot recognise the presence of underlying continuous deterioration.

2.4.2 Percentage Transition method

A number of studies have used the percentage transition method, which indicates a change in road condition state from the previous state. For example, Pierce (2003) used the rigid pavement to derive RD models (IRI vs. time) using 5-years data. The IRI summary table was used to obtain a TPM, and the road deterioration trend was derived using a Monte Carlo simulation. This method is known as the percentage transition method. The major limitations of that study were i) no use of any asphalt pavement, and ii) the results were not valid for the network level as only one rigid pavement was considered. Tack and Chou (2002) used 4-years data to obtain a TPM, and a Monte Carlo simulation and statistical analysis to obtain the RD models. Similarly, Jin, et al. (2010) and Mandiartha (2010) developed a homogeneous TPM for the roughness-based road deterioration modelling.

Panthi (2009) used the 2-stage non-homogeneous TPMs to develop RD models for cracking, rutting and roughness with the help of condition distribution data and percentage transition analysis. Ranjith, et al. (2011) used this method to compare with minimum error statistically, and found that the percentage transition method did not always outperform the minimum error method. However, their study was done to predict bridge deterioration and no RD simulation was provided in their analysis.

The transition probability of each pavement type is estimated by using the following formula.

$$\mathrm{P_{ij}} = \frac{\mathrm{N_{ij}}}{\sum_{\mathrm{k}=1}^{5} \mathrm{N_{ik}}} \qquad \mathrm{i, j} = 1,2,3,4,5 \tag{2.7}$$

Where P_{ij} is the transition probability that state i is changed to state j in the following year; N_{ij} is the number of transitions (from state i at time t to state j at time t+1). Therefore, the TPM is composed of all transition probabilities derived using the following equation.

$$\mathrm{P} = \begin{bmatrix} p_{11} & p_{12} & p_{13} & p_{14} & p_{15} \\ p_{21} & p_{22} & p_{23} & p_{24} & p_{25} \\ p_{31} & p_{32} & p_{33} & p_{34} & p_{35} \\ p_{41} & p_{42} & p_{43} & p_{44} & p_{45} \\ p_{51} & p_{52} & p_{53} & p_{54} & p_{55} \end{bmatrix} \tag{2.8}$$

The Percentage Transition Method is recommended to derive a non-homogeneous TPM as it can address explanatory variables and inherent pavement deterioration.

2.4.3 Probit model

The probit model is a type of discrete choice model where the dependent variable can only take two values. The model is most often estimated using the standard maximum likelihood procedure. Li (2005) proposed the ordered probit model and sequential logit model for the probabilistic RD modelling. The proposed models were tested with the AASHTO road test data and compared with the Markov chains, and found to be superior. Moreover, the structural state space (real time) model was offered for assessing the dynamic nature of pavements deterioration to update the RD models so that they were consistent with real road data. However, the study did not use any field data for model development. Although the sequential logit model had similarities to the transition probabiliy, no TPM was developed.

Madanat, et al. (1995) used the ordered probit technique for developing the non-homogeneous transition probabilities. They considered the incremental deterioration models, and checked accuracy of the model using verification techniques. The results showed that the proposed method was more realistic as it recognised the hidden nature of pavement performances and explicitly linked it to the explanatory variables.

However, Fu and Debraj (2008) identified that the ordered probit model was not always suitable as it needs large amount of data, and may also be based on panel data. Moreover, it assumed that the observed condition states were independent and had similar distributions (Madanat, et al. 1995; Madanat and Wan-Ibrahim, 1995).

2.4.4 Conversion from Deterministic model

Li, et al. (1996); Li (1997) and Li and Haas (1998) used a conversion method to derive a TPM from a deterministic equation. The research used a Monte-Carlo simulation to generate the random variables for developing the TPM and transition state vectors. The Bayesian technique was utilised for validation of the derived probabilistic models. Furthermore, these studies checked the sensitivity of traffic growth rate, subgrade deflection and pavement thickness in deriving TPMs as they were based on the pavement design equation. Li, et al. (1995) mentioned that conversion to a probabilistic RD model from a deterministic equation is an easy approach compared to engineering judgment and the use of historical data.

However, the research mainly concentrated on the flexible pavements. In addition, this method has been rarely used by others as it considers conversion of an existing model rather than deriving a new one.

2.4.5 Bayesian technique

The Bayesian statistics provide evidence on the true state in terms of degrees of belief, or more specifically Bayesian probabilities. The Bayesian approach aims to obtain realistic parameter distributions through knowledge and updated data (Hong and Prozzi, 2005). Hong and Prozzi (2005 and 2006) used this method to address heterogeneity of individual model parameters. The Gibbs sampling algorithm and the Monte Carlo Markov Chain (MCMC) were utilised to estimate probabilistic parameters distribution and density functions. However, the study used the American Association of State Highway Officials (AASHO) road test data and did not collect new field data. Another study considered the Kansas road data using the Bayesian approach to account for heterogeneity of pavement roughness, and then used the hierarchical MCMC simulation for deriving RD models (Mills, 2010).

The Bayesian approach was utilised for validation of the developed RD models with field data rather than developing the models (Li, 1997). FHWA (1990) and Panthi (2009) stated that the method may not consider mechanistic behaviour of a pavement. Moreover, it heavily depends on the data and improper judgment could lead to erroneous models.

2.4.6 Multi-stage Hazard model

A hazard model is a statistical technique for determining 'hazard functions', or the probability that an individual will experience an event within a particular time-period, given that the individual is subject to the risk that the event might occur. The hazard model was first used in confidence analysis. Quite a few studies have used it for the RD modelling, for example, Eguchi, et al. (2006) for a homogeneous TPM and Lethanh and Adey (2012) for a non-homogenous TPM. Eguchi, et al. (2006) developed the condition index and rutting deterioration models with the help of the multi-stage hazard model for a homogeneous TPM. Kobayashi et al. (2010) used an exponential hazard model to derive a non-homogeneous TPM. The maximum likelihood method was undertaken using the pavement structural strength and loading data to determine parameters of the exponential hazard model. It was proposed to review more data for deriving appropriate parameters.

Mishalani and Madanat (2002) used the Weibull hazard rate model to derive a non-homogeneous transition probability. They proposed a time-based discrete-state stochastic duration model based on the Weibull hazard distribution using the reinforced concrete bridge deck data. In the study, pavement structural number and loading data were utilised by the maximum likelihood method to determine parameters of the exponential hazard model. However, the study's conclusion did not support the results, and it proposed a review of more data for deriving appropriate parameters. Lethanh and Adey (2012) used a multi-stage exponential hazard model to derive a TPM when the data set was incomplete.

The major limitation of the Hazard model are that it assumes a two-stage transition of the condition state only, which means it is not possible to consider the normal deterioration as well as a flood induced deterioration.

The assumptions and limitations of the above-mentioned six approaches in developing RD models are given in *Table 2.2*.

Table 2.2: Different methods of Markov Chain

Broad Area	Key Feature	Methods Used	Sources	Assumptions	Limitations
Transition Probability	Shows transition in condition state.	Ordered probit and sequential logit models*	(Fu and Debraj, 2008; Li, 2005; Madanat, et al. 1995; Madanat and Wan-Ibrahim, 1995)	Probit model considers one variable as independent to study change in condition state. Sequential logit model has similarity with a non-homogeneous TPM. Can address latent nature of pavement performance and links road deterioration to explanatory variables.	Does not provide transition matrix. Needs large amount of data and may need panel data. Real field data is not utilised. Assumes observed independent condition states and similar distribution. Considers pavement heterogeneity.
Homogenous TPM	Transition in condition state with matrix form and not related to time.	Percentage transition	(Jin, et al. 2010; Mandiartha, 2010; Panthi, 2009; Pierce, 2003; Robinson, et al. 1998; Wang, et al. 1994)	A TPM is derived using probability of transition from one state to the next.	Uses real data to derive a TPM. Can address explanatory variables and inherent pavement deterioration.
		Chapman-Kolmogorov equation	(Li, et al. 1996; Wang, et al. 1994)	A mathematical technique related to the Markovian stochastic process. Can examine a TPM.	Cannot check pavement inherent deterioration.
		Minimum error objective function	(Ortiz-Garcia, et al. 2006; Wu, 2008)	Objective is to have minimum difference between the predicted and observed values of a specific state of the TPMs. Based on the historical data and engineering experiences.	Does not capture various explanatory variables. Non-homogeneity is not considered properly. Presence of underlying continuous deterioration not recognised.
		Statistics & Monte Carlo simulation	(Tack and Chou, 2002)	Uses statistics and a Monte Carlo simulation as tools. Used when less data are available.	An analysis tool.
		Exponential hazard model	(Eguchi, et al. 2006; Shin and Madanat, 2003)	It shows the rate that relates to duration.	Assumes one type of condition state' transition. Assumes similar road structure for all sections. Cannot capture duration dependence properly.
		Gaussian elimination	(Tjan and Pitaloka, 2005)	It is an algorithm for solving systems of linear equations, which can find the rank of a	An analysis tool.

				matrix, calculate the determinant of a matrix and derive the inverse of an invertible square matrix.	
Non-homogeneous TPM	Transition in condition states with matrix form, and related to time.	Percentage transition*	(Panthi, 2009; Pierce, 2003; Ranjith, et al. 2011; Tack and Chou, 2002)	A TPM is derived using probability of transition from one state to the next.	Uses real data to derive a TPM. Can address explanatory variables and inherent pavement deterioration.
		Conversion*	(Hudson, et al. 1998; Li, 1997; Li and Haas, 1998; Li, et al. 1995; Li, et al. 1998)	Easier and quick comparison to typical RD modelling.	This method and statistical approach provide similar results with negligible difference.
		Exponential hazard model*	(Kobayashi, et al. 2010; Lethanh and Adey, 2012; Mishalani and Madanat, 2002)	Suitable for incomplete data set.	Assumes transition of condition state as perpendicular. Assumes similar road structure for all sections. Cannot capture duration dependence properly. More data review is needed for deriving appropriate parameters.
		Bayesian*	(FHWA, 1990; Hong and Prozzi, 2005; Hong and Prozzi, 2006; Li, 1997; Li, et al. 1998; Mills, 2010; Panthi, 2009)	Can be used for model calibration. Aims to get realistic parameter' distribution through knowledge and updated data. Can address heterogeneity of individual model parameters.	No new real data used. Analysis tool. May not consider mechanistic behaviour of pavement. Heavily depends on the data. Improper judgment can lead to erroneous model.
		Reliability analysis	(Li, et al. 1996)	For assessing probability of a system completing its expected function during an interval of time.	An analysis tool.
		Minimum error objective function*	(Butt, et al. 1994; Chun, et al. 2012; Fu and Debraj, 2008; Madanat, et al. 1995; Ortiz-Garcia, et al. 2006; Shafahi and Hakhamaneshi, 2009)	Objective is to have minimum difference between predicted and observed values of a specific state of the TPM. Based on historical data and engineering experiences. Found useful in case studies for bridge and rail.	Does not capture various explanatory variables. Non-homogeneity is not considered properly. Presence of underlying continuous deterioration not recognised.
		Monte Carlo simulation and reliability analysis	(Mills, 2010)	Reliability analysis using a Monte Carlo Simulation.	The study results did not match with treatment effects.

*These are the key techniques for non-homogeneous probability transition and TPM. Monte Carlo simulation and reliability analysis are general techniques.

2.5 Work Effect (WE) Models

The WE means a change in road condition due to a maintenance treatment, which can be addressed through the WE models. Several methods have been used in deriving treatment WE models, for example, deterministic (Lu, 2011; Wang, et al. 2003; Wang and Goldschmidt, 2008), Markov Chain with a homogeneous TPM (Akyildiz, 2008; Jin, et al. 2010; Morcous and Lounis, 2005) and Markov Chain with a non-homogeneous TPM (Abaza and Ashur, 1999; Butt, et al. 1994; Garza, et al. 2011; Li, et al. 1998; Panthi, 2009). In reality, the effect of a treatment means the transition from fair/bad/worse to good condition, which is normally one transition after a treatment. Therefore, both homogenous and non-homogeneous TPMs represent real field conditions.

2.6 Impact of Mr Loss on Pavement Performance

A pavement's performance deteriorates with time, which is dependent on traffic loading, material properties (pavement type, structure, strength and subgrade strength), climate/environment, drainage, initial roughness value and maintenance activities (Hunt and Bunker, 2001). Both the pavement design and performance relationships are linked to the material properties which may be represented with the Mr of pavement layers. It has been observed that modulus of subgrade and granular materials are dependent on stress and moisture content (Finn, et al. 1977; Khan, 2005; Monismith, 1992; Nataatmadja, 1992; Ullidtz, 1979); on the other hand, modulus of bituminous material is sensitive to temperature, vehicle speed and loading time (Finn, et al. 1977; Khan, 2005; Van der Poel, 1954).

In fact, the modulus of granular and subgrade layers reduce significantly when moisture is entered (Brown and Dawson, 1987; Drumm, et al. 1997; Yuan and Nazarian, 2003). Monismith (1992) stated that a lower modulus leads to increased pavement deflections and consequently a reduced pavement life, which was also supported by Huang (1993). In view of the above, it can be considered that Mr influences on the pavement design and performance.

The loss of Mr at different pavement granular and subgrade layers due to extreme moisture intrusion can represent a flood. A road structure may be inundated for several days after a flood; and as a consequence, it becomes weaker with very low Mr at untreated layers and may considerably lose shear strength. For example, Rokade, et al. (2012) and Amiri, et al. (2009)

mentioned that excessive water in a pavement causes early distress, and structural or functional failure. They identified the following four consequences due to a flood and/or very poor drainage.

- Reduction of subgrade and base/sub-base strength;
- Differential swelling in expansive subgrade soils;
- Striping of asphalt pavement; and
- Movement of fine particles into base and sub-base course materials which significantly reduces hydraulic conductivity.

Therefore, transmission of dynamic loads through different layers of a pavement is affected during flooding (Moulton, 1980). Generally, a flooded road would experience on an average about 15 times more damage compared to a good drained section (Yuan and Nazarian, 2003).

As an example, the impact of the hurricanes Katrina and Rita are highlighted here. Helali, et al. (2008) covered their impact on the pavement performances, which has been shown in Chapter One. They found poor responses from the flood damaged roads. Similarly, Zhang, et al. (2008) assessed roads after the flood due to the hurricane Katrina in 2005 and then Rita, mainly at New Orleans of USA. In total, 3,220 km roads were flooded for 5 weeks. They considered 383 km flooded and non-flooded roads as sample. The initial investigation results indicated a 15% reduction of SN for the AC pavements which was only 5% for the concrete pavements. The flexible pavements' SN were estimated using the AASHTO method, while a modified SN value as SN_{eff} was used for the concrete pavements. The AC pavement experienced 20% subgrade Mr loss and 46% increase in deflection; whereas, concrete pavements had only 1% subgrade Mr loss and 9% increase in deflection. In general, the study found that the flooded pavements had lower subgrade Mr which affected on deflection and SN. A rigid pavement performed better than the AC and composite (asphalt surface on concrete) pavements. The study found that a flooding duration even for one week is adequate to cause the total damage, and there is not any major damage after this period; which was also agreed upon by Yuan and Nazarian (2003). Again, it was observed that the thinner AC pavements were more adversely affected than the thicker ones.

Gaspard, et al. (2006) assessed after flood data for the hurricane Katrina of 2005 at New Orleans, and found the followings:

- the thinner asphalt became weaker than the thicker one;

- very little damage was detected for the rigid pavements, which was also valid for the composite pavements;
- flooding duration beyond 7 days did not have any further damaging impact to the pavements, as they were already severely damaged;
- for an after flood structural evaluation, the SN, deflection and Mr data are necessary along with the IRI, rutting and cracking data; and
- the composite pavement needs 25 mm of AC as rehabilitation, whereas the thin pavement needs minimum 75 mm of asphalt overlay to enhance SN.

The AASHTO 2008 has included an Enhanced Integrated Climatic Model (EICM) for obtaining Mr values at granular layers due to moisture inclusion (Cary and Zapata, 2010; Zapata, et al. 2007), which is given below.

$$Mr = F_{env} * M_{ropt} \tag{2.9}$$

Influence of moisture on the Mr of unbound materials can be derived as below:

$$\log F_u = \log (Mr/M_{ropt}) = a + (b - a)/(1 + \exp (\ln (-b/a) + (k_m * (S - S_{opt})))) \tag{2.10}$$

where,

Mr = modulus of resilience of a specific layer

M_{ropt} = modulus of resilience of a specific layer at optimum moisture content

F_{env} = influence of climate/environment

F_u = influence of climate to unbound granular layers

a = minimum of log (Mr/M_{ropt})

b = maximum of log (Mr/M_{ropt})

k_m = regression parameter

S = degree of saturation

S_{opt} = optimum moisture/saturation

There was a severe flood in Queensland, Australia in 2010-11, which effect has been discussed in Chapter One. Faturechi and Miller-Hooks (2015) reviewed almost 200 recently published papers since 90s that discussed the performance of transportation infrastructure in disaster, where it was presented using risk, vulnerability, reliability, robustness, flexibility, survivability and resilience. However, no indication of a pavement performance after a flood was given.

The above results show the impact of a flood on road assets. The modulus of pavement granular and subgrade layers are reduced tremendously due to high moisture content during a flood; and as a result, the deflection value increases and the SN reduces. In addition, if high traffic movement is allowed, it would worsen the scenario. An after flood pavement structural analysis would be useful for future planning purposes.

The current study has aimed to investigate the impact of change in Mr due to moisture increase on pavement performances during flooding. Generally, flooding, heavy rainfall with water ponding, localised flooding and poor sub-surface drainage may increase moisture in pavement layers. Natural disaster like flooding has a huge effect on the road assets, and hence it has to be taken into account in the analysis. It is believed that this will help understanding the behaviour of a road structure during flooding, as loss of Mr due to moisture intrusion is a typical scenario.

Although, Helali, et al. (2008) and Zhang, et al. (2008) estimated pavement damage after a flood, no study so far addressed pavement performances during flooding or suggested a prediction model. Ullidtz (1998) used two existing models, i.e., AASHTO and HDM-III models, to check pavement performances at no flood condition. He found that the rate of road deterioration is function of loading for the AASHTO case and dependent mainly on the pavement age for the HDM-III case. However, this is not valid in all cases, as pavement deterioration is related to traffic loading along with material properties, environment, drainage, and maintenance activities. The analysis did not cover a natural disaster like flooding scenario on pavement responses and performances. It is suggested using the AASHTO 2008 and HDM-4 models to predict pavement performances during flooding using the change in roughness.

2.7 Flood-resilient Pavement

A flood-resilient pavement can sustain flooding better, which can be determined by investigating pavement performances after a flood. Some studies assessed pavement responses after flooding (Helali, et al. 2008; Zhang, et al. 2008; Yuan and Nazarian, 2003). However, they used the deflection data for the analysis, and did not cover pavement performances. In addition, their road grouping was only based on the pavement types. As a result, a detailed investigation is needed to address pavement performances with flooding.

Generally, raised pavement, adequate drainage structure, proper sub-surface drainage facilities, appropriate materials and stable embankment slope can ensure better performance of a road with a flood. All these solutions are outside the scope of the research; rather, pavement strengthening with AC overlay and/or stabilisation of granular and subgrade layers are considered here for ensuring a flood-resilient pavement. Recent studies show that a thick AC overlay increases pavement strength as a post-flood solution (Helali, et al. 2008; Yuan and Nazarian, 2003). Zhang, et al. (2008) observed that a rigid pavement performs better with a flood. Therefore, it is concluded that providing a strengthening overlay with or without granular layers stabilisation to a road before a flood can potentially make it stronger against a flood. This can also be done by converting a flexible pavement into a rigid one.

2.8 Optimum M&R and Pre- and Post-flood Road Maintenance Strategies

Although some road authorities have maintenance strategies, they are not always the optimal solution. Current literature does not highlight any optimum strategy that can address pavement life cycle performances incorporating flooding. Therefore, this section has targeted overcoming the limitation through developing sound and optimum M&R, pre- and post-flood road maintenance strategies.

An M&R strategy ensures appropriate and timely maintenance of a road network. The general maintenance policy is the right treatment at the right time and at the right place, which is achieved through an appropriate M&R strategy. However, the strategy has to be optimum in order to ensure efficient use of an allocated budget. A PMS should have a sound M&R strategy to preserve its road assets efficiently.

Several studies (Akyildiz, 2008; Chikezie, et al. 2011; Garza, et al. 2011; Li and Kumar, 2003a) have discussed optimisation algorithms using mainly the Linear Programming (LP), Non-linear Programming (NLP), Integer Programming (IP), Dynamic Programming (DP), Goal Programming (GP), Genetic Algorithm (GA), Artificial Neural Network (ANN) and Fuzzy logic to obtain an optimal solution. Detailed features of these are shown in *Table 2.3*.

It appears that the LP, NLP, IP and DP are suitable for a network level analysis although they cannot always handle large number of variables. A LP has been successfully used in different studies (Abaza and Ashur, 1999; Akyildiz, 2008; Garza, et al. 2011; Hudson, et al. 1998; Liu and Wang, 1996; Wang, et al. 2003). Another popular technique is an IP (produces similar results as LP using decision variables with 0 and 1), which has effectively been used in a range of diverse studies (Scheinberg and Anastasopoulos, 2010; Li, et al. 1998; Hiep and Tsunokawa, 2005; Sirirangsi, et al. 2003; Fwa, et al. 1998; Porter, 1987; Wang and Goldschmidt, 2008). It is worth mentioning that the HDM-4 model uses an IP to obtain an optimum M&R strategy (Odoki and Kerali, 2000). Although the GA and ANN can work with multiple constraints, but they do not always produce the true optimal solutions (see *Table 2.3*). On the other hand, a DP finds a solution by starting at the final condition and working backwards.

It is recommended using the HDM-4 model for developing pre- and post-flood maintenance strategies and optimum M&R strategy considering pavements' LCA. The HDM-4 is a well-known economic tool and widely used in different parts of the world including Australia. Odoki and Kerali (2000) proposed 'minimising agency cost and maximise performance target (at set IRI) with budget constraints' – as the optimisation objective in this model, which is a useful one for managing assets with constrained budget. The HDM-4 model uses an IP for optimisation that is suitable for a network level analysis and can address the decision variables. The IP algorithms shown in Odoki and Kerali (2000) are as follows:

Maximising the Total Objective Function (TOBJ) for the network:

$$Maximise\ TOBJ[Xsm] = \sum_{S=1}^{S} \sum_{m=1}^{Ms} OBJsm * Xsm \quad (2.11)$$

where,

s = a road section (s = 1, 2, …, S)

S = total sections

M_s = the number of alternatives for road section s

m = an investment alternative on a road section

OBJ_{sm} = the objective function to be maximised which may be the discounted net present value of economic benefits, or the average reduction in roughness due to the investment alternative.

X_{sm} = the zero-one decision variable:

= 1 if alternative m of investment unit s is chosen

= 0 otherwise

$m = 1, \ldots, M_s$

the above is subject to the following resource constraints:

$$\sum_{S=1}^{S}\sum_{m=1}^{Ms} Rsmqt * Xsm \leq TRqt \qquad q = 1, \ldots, Q;\ t = 1, \ldots, T \qquad (2.12)$$

where,

R_{smqt} = non-discounted amount of resource type q incurred by the sectoral agency within a budget period t

TR_{qt} = maximum amount of resource type q available for budget period t

Q = the total number of resource types

T = the total number of budget periods

The above is subject to the constraint of mutual exclusivity:

$$\sum_{m=1}^{Ms} \mathrm{Xsm} \leq 1 \qquad s = 1, \ldots, S \qquad (2.13)$$

that is, for each road section s, no more than one alternative can be implemented. If M is the average number of alternatives for the roads, the problem then has SM (= S*M) zero-one variables, QT (= Q*T) resource constraints and S interdependency constraints. The parameters that define the problem size are S, M and QT.

Table 2.3: Comparison between optimisation algorithms

Method	Sources	Features	Advantages	Disadvantages
LP	(Kuhn and Madanat, 2005; Li and Kumar, 2003b)	• Objective functions and constraints are formulated as linear equations. • Decision variables are continuous. • Most common method used in a PMS.	• Simple • Suitable at the network level	• Suffers from combinatorial explosion problems • Difficulty in maintaining the identity of individual pavement sections
NLP	(Li and Kumar, 2003b)	• Objective functions and constraints are formulated as non-linear equations.	• Suitable at the network level	• Cannot handle a large number of decision variables • Suffers from combinatorial explosion problems • Difficulty in maintaining the identity of individual pavement sections
IP	(Li, 1997; Li, et al. 1998; Li and Kumar, 2003b; Lu, 2011; Wu and Flintsch, 2008)	• Objective functions and constraints are formulated as linear and non-linear programming. • Decision variables use integer or whole number (0 or 1) values.	• Suitable at the project level or project-based network analysis	• Cannot handle a large number of decision variables
DP	(Li and Kumar, 2003b; Mbwana and Turnquist, 1996)	• No existing standard mathematical formulation. • The problem is divided into stages, with a decision required at each stage. • Each stage has a number of states associated with it. The effect of a decision is to transform the current state to a state associated with the next state. • The solution procedure is to find an optimal policy for the overall problem.	• Applicable in making a sequence of interrelated decisions, e.g., multi-year budget optimisation • Reduced computational complexity • Suitable either for the network level or project level analysis	• Cannot handle a large number of decision variables
GA	(Chan, et al. 2003; Li and Kumar, 2003b)	• Based on natural selection and natural genetics. • Through continuous copying, swapping, and modifying of partial strings which are generated in an initial pool of solutions, to allow the solution pool to evolve toward better solutions.	• Capable of solving combinatorial problems • Can handle a large number of decision variables • Flexible in defining the objective function	• Does not generate a true optimal solution
ANN	(Li and Kumar, 2003b)	• The model is composed of a large number of nodes. • Each node is associated with a state variable and an activation threshold. • Each link between nodes is associated with a weight. • State of node is determined by an activation function.	• Capable of solving combinatorial problems • Can handle a large number of decision variables	• Does not generate a true optimal solution

2.8.1 Derivation of optimum maintenance standards

A maintenance standard is a set target to manage a road efficiently in its life cycle, which also represents the allowable limit of road deterioration (Odoki and Kerali, 2000). A standard is based on road surface class, characteristics of traffic and general operational practice (Odoki and Kerali, 2000). Generally, when roughness reaches the standard (fixed with the IRI), a treatment is required to restrain road roughness beyond that standard. Standards have to be optimum considering cost and road condition, and should be set at the network level. Maintenance strategies are a long-term plan which should be implemented so that a road network is kept in good condition. They can help decision makers understand the road network and budget scenario.

Several studies have described procedures for setting maintenance standards. They are based on road grouping with representative roads, maintenance trigger levels and monitoring (ASRA, 1980). The Road Infrastructure Maintenance Evaluation Study (RIMES) study explains the steps in detail (RIMES, 1999), which are shown below.

- Set up categories and traffic definitions,
- Determine available parameters for class definitions, and define classes based on available parameters,
- Set up trigger levels corresponding to class limit definition,
- Define feasible (based on engineering considerations) maintenance actions for each pavement/bridge class,
- Estimate costs,
- Use deterioration models,
- Set budget constraints,
- Set condition constraints,
- Establish functions to be optimized, and
- Produce maintenance standards.

Many studies have used the HDM-4 model to derive optimum maintenance standards (Tsunokawa and Ul-Islam, 2003; Jain, et al. 2005; Jain, et al. 2007; Khan, 2006; Khan and Odoki, 2010). The HDM-4 was developed by the World Bank and other international organizations. It is an economic tool for conducting life-cycle cost-benefit analysis that can also predict pavement performances

(ISOHDM, 2003). In Malaysia, for its 28,000 km road system, standards were determined considering engineering judgment, custom and practice of the Malaysian Public Works Department. Again, in Cyprus with less than 2,000 km of road system, standards were developed using the HDM-III model (Snaith, et al. 1994), which is similar to the RIMES approach of using decision-making tools. Some other recent studies developed both roughness-based optimum pavement maintenance standards and a long-term strategy for 21,000 km of major roads in Bangladesh (Khan, 2006; Khan and Odoki, 2010; Khan, 2012). The optimum maintenance standard for each road group was analysed using the HDM model with different combinations of maintenance interventions at different maintenance standards, and the resulting economic indicators were compared. The optimization objective was to 'minimise total agency cost, maximise benefits or reduce roughness values'.

Procedures to develop optimum maintenance standards using the HDM-4 model are as follows (Odoki and Kerali, 2000):

- Categorize road network into matrix cells;
- Define representative traffic volume and loading for each cell;
- Define maintenance and improvement standards;
- Specify budget constraints;
- Model pavement deterioration for each matrix cell;
- Apply different maintenance and improvement standards;
- Analyse road user costs; and
- Select the optimum maintenance and improvement standards.

The HDM-4 model was used to derive maintenance standards for low volume roads and multilane highways, respectively, in India (Jain, et al. 2007; Jain, et al. 2013). These studies considered treatment alternatives for one specific IRI, and hence no optimum standards were derived. Rather optimum strategies were chosen for a given IRI. In their analyses, Khan (2006) and Khan and Odoki (2010) used 48 road groups using two types of surface, three types of traffic volume and eight types of pavement width. Although these studies derived optimum standards, they excluded important parameters, i.e., pavement strength and traffic loading, that effect on the pavement performances.

It is worth mentioning that the Cooperative Research Centre for Construction Innovation (CRC CI) undertook a detailed study in deriving maintenance strategies for the TMR-QLD using 4,500 km of roads as a sample (CRC CI, 2006). It used the HDM-4 model for obtaining the results. However, the analysis did not set optimum economic maintenance standards for the road groups, rather these were proposed after statistical analysis of the HDM-4 results. The study suggested compound treatments as a strategy which also included a reconstruction. Generally, a reconstruction is used as a base case along with different alternatives for optimum treatment selection; but it is not desirable in asset management practice as it allows complete deterioration of a road without providing any preventive and/or periodic maintenance in the life cycle. Similarly, Khan (2014) developed a maintenance strategy for managing rural roads in Botswana, but did not set optimum maintenance standards. Some other studies discussed the development of maintenance strategies for the TMR-QLD and other major Australian roads (Cancian, et al. 2013; Smith and Cerecina, 1998). However, those studies did not generate detailed results and optimum strategies.

2.8.2 Derivation of pre- and post-flood strategies

Many studies derived a road maintenance strategy for normal condition (Tsunokawa and Ul-Islam, 2003; Jain, et al. 2005; Jain, et al. 2007; Jain, et al. 2013; Khan, 2006; Khan and Odoki, 2010; Snaith, et al. 1994). No life-cycle road maintenance strategy around the world includes flooding events. This lack has an impact on budget and pavement performances. A life-cycle maintenance strategy with flooding may be of either pre- or post-flood types. A road authority can include RD prediction models with flooding while deriving optimum pre- and post-flood maintenance strategies reflecting a changed trend in road deterioration.

A post-flood strategy reveals a pavement’s life-cycle performance with a flood, and recommends for necessary treatments as post flood rehabilitation. The asphalt overlay and/or granular and subgrade layers stabilization could be used as a post-flood treatment. The HDM-4 optimisation algorithm is found suitable for deriving pre- and post-flood road maintenance strategies.

It is assumed that a pre-flood road maintenance strategy for a road authority can potentially improve network resilience against flooding and save a huge cost of post-flood treatments. The analysis assumes a treatment in year 1 as a part of pre-flood strategy to increase pavement strength so that it becomes a strong road. As a result, a pavement would perform better in its life-cycle if a flood comes in future. The pavement also performs better in the normal condition as a result of this investment.

The new roughness and rutting based RD models with flooding may be used to predict road deterioration after a flood. The approach and results would provide a better road management with realistic maintenance strategies.

2.9 Flood Risk Assessment

A risk assessment shows how vulnerable a road is after a disaster, in this case a flood event. Risk analysis is based on likelihood of an event and its consequences, as given below.

Risk = Likelihood * Consequences (AS/NZS, 2009) (2.14)

Generally, engineering judgment is essential for defining likelihood and consequences. Several studies used risk factor in a PMS (Paine, 2004; Reigle and Zaniewski, 2005). Paine (2004) used a risk analysis for asset preservation; however, he did not consider life-cycle economic analysis. On the other hand, Reigle and Zaniewski (2005) used risk as a factor in the life-cycle analysis for developing cost-effective pavement preservation strategies, but did not do any risk assessment. Therefore, these studies did not address a comprehensive risk analysis for roads. The TMR-QLD utilizes a risk analysis model named RISKO in its PMS, which uses the outputs provided by a PMS tool (TMR, 2012b). This is a MCA technique, but misses' scientific logic to provide weight on likelihood and consequences. It considered safety and life-cycle costs for risk analysis, but omitted pavement performances, loading and disaster.

A detailed risk analysis was done on the performances of unbound granular materials. The study considered traffic, rainfall, material properties, cross-section design, construction and maintenance for estimating likelihood; and consequences were rated using failure mode like rutting, potholes, cracking and skid resistance (Creagh, 2004). A fault tree technique using Boolean logic (AND and OR) was used to identify the key failures from different sub-failure modes in calculating risks in order to obtain the semi-quantitative risk ratings on likelihood and consequences (Creagh, 2004). Several other studies also considered the fault-tree technique for determining causes and sub-causes of each failure as consequences to score a rating (Creagh, 2006; Williams, 2002; Williams, et al.

2001). Creagh (2004) undertook calibration and validation of risk results with actual performances for some roads, and found acceptable. The following limitations were observed in that study:

- usable for low traffic volume of roads;
- very subjective;
- valid for materials analysis on risks; and
- risk cost-effectiveness was not used in the road asset management.

A few studies deal with a flood risk assessment on roads. Apart from the above, some other studies considered vulnerability of flood risk, but they were not detailed and did not address pavement performances after a flood. These studies assumed vulnerability analysis with different flooding consequences such as traffic congestion, blockings, pavement damage and its costs, etc. (Karmakar, et al. 2010; Benedetto and Chiavari, 2010; Prina, et al. 2004; and Bengtsson and Tomasson, 2008). However, no analysis covered a consequence evaluation before and after a flood to get a concrete result with and without a flood. Moreover, they considered a very small portion of a network with limited data. As a result, it can be said that no in-depth flood risk assessment on pavement performances was addressed before.

A flood risk assessment is important for managing roads as a proactive approach before a flood. Therefore, a risk analysis has to be done cautiously in finalizing likelihood and consequences factors. It is believed that a flood risk assessment of roads would help obtaining cost-effective decisions for sustainable pavement management.

2.10 Summary of the Overall Review

This chapter did a thorough literature review on RD models; pavement responses after a flood due to Mr loss; maintenance standards and strategies, optimisation for obtaining M&R, pre- and post-flood strategies; and risk assessment. The current research gaps are identified and highlighted.

Presently, flood damaged roads are assessed based on after flood inspection when water recedes. Therefore, the pavement performances with flooding are not captured. A limited number of road deterioration models with flooding are available. However, they have major shortcomings, i.e., i) they are either regression based or deterministic models, ii) do not have a provision for long term deterioration prediction after a flood, iii) did not consider major pavement types, iv) no project level or site-specific RD models with flooding were derived, v) road grouping was simplistic, vi) pavement performance with two key factors (roughness and rutting) were not directly addressed, and vii) no simulation was considered to predict road deterioration at different probabilities of flooding.

As a result, the existing literature review reveals a drawback for appropriate, probabilistic and simulation-based project and network levels RD models with flooding. Pavements' resilience due to flooding needs to be explored. Moreover, there are not any optimum maintenance strategies that can address flooding in LCA. A logical flood risk assessment on pavement performances is also required.

The literature review has covered major components of a PMS. It has highlighted the current PMS being used in different major road authorities of Australia. Importance and types of RD models (deterministic and probabilistic models) were discussed. In general, pavement deterioration is dependent on several key factors, that is, material characteristics, loading and environment; all of which are stochastic in nature. Therefore, it is rational to use probabilistic models for the RD modelling.

Details of the RD models based on Markov chain including homogeneous and non-homogeneous TPMs were highlighted. It has been observed that a non-homogeneous TPM can show real road deterioration, as deterioration probability is a function of time. Six methods have been identified for the RD modelling with non-homogeneous TPMs, i.e., minimum error method, percentage transition method, probit model, conversion from deterministic model, Bayesian technique and multi-stage hazard model. After detailed investigation, the percentage transition method was found suitable.

Impact of Mr losses on the pavement responses have been highlighted from several studies. Generally, a pavement gets weaker after a flood due to Mr loss at granular layers, which effect in decreasing the deflection and SN values. It is found that pavement performances due to Mr losses after a flood could be assessed using the AASHTO and HDM-4 roughness models. These results would be useful in strategic and sustainable planning.

An optimum maintenance strategy would help managing assets efficiently in life cycle. Different methods on setting maintenance standards and strategies are highlighted. In addition, several optimisation techniques (LP, NLP, IP, DP, GA and ANN) and objectives are shown for developing the pre- and post-flood strategy and optimum M&R strategy. Finally, an IP has been found suitable with proposed optimization objective as ‘minimise agency cost and maximise performance target at budget constraints’. The HDM-4 model has been found useful for life-cycle cost-benefit analysis, which optimisation algorithms have been highlighted.

Apart from different techniques that are discussed, a flood risk assessment is also found suitable for addressing different types’ pavements performances after a flood.

Finally, the literature review gives recent knowledge gaps on a PMS including flooding, and has provided adequate information for developing the research approaches.

CHAPTER THREE: METHODOLOGY

This chapter describes approaches used in deriving the roughness and rutting-based RD models with flooding, impact of Mr loss on pavement performances, flood-resilient pavements, pre- and post-flood road maintenance strategies, optimum maintenance standards, and flood risk assessment. The overall analysis approach is shown in *Figure 3.1*. The RD models with flooding are derived at different probabilities of flooding at first, which are validated with the actual data, a t-test, the AASHTO and HDM-4 roughness models. Assessing a difference between two sets of data with small sample sizes needs a tool like t-test, and hence it was used to check the variances between the RD models and actual data. The optimum road maintenance strategy, pre- and post-flood strategies are derived using the derived RD models after a flood (if needed) and the HDM-4 optimisation algorithm. The two new indicators, i.e., ΔIRI/Pr and ΔIRI/MrL, are used along with the RD modelling results to get the flood resilient pavements. A flood risk assessment has been done using the flood consequences results obtained from the RD models. All these help improving a PMS with flooding. The entire approaches are discussed below.

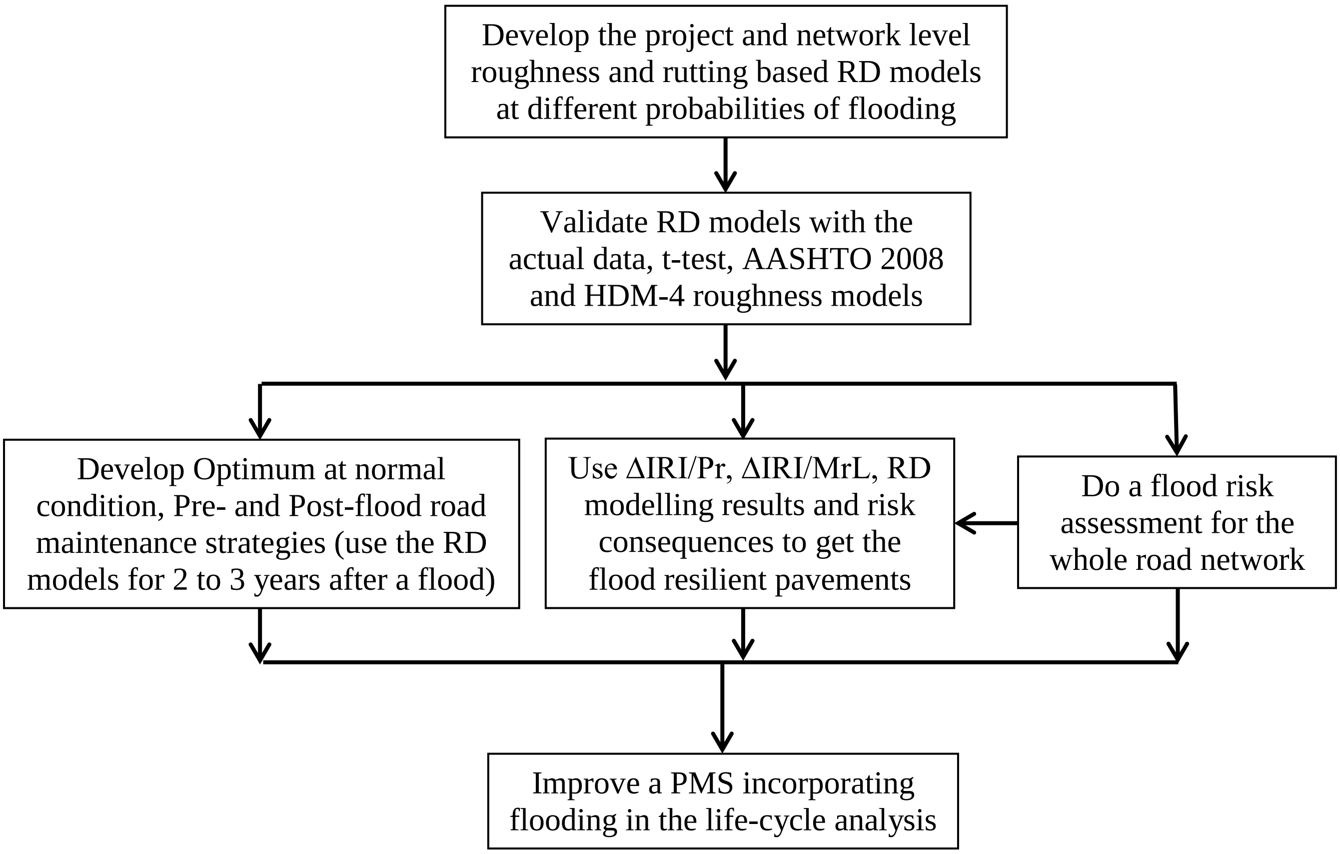

Figure 3.1: Overall approach of analysis

3.1 Development of the Roughness and Rutting-based RD models with Flooding

The proposed probabilistic roughness and rutting-based RD models have been generated with non-homogeneous TPMs using the percentage transition method. In the analysis, a pavement's performance with flooding is represented with the roughness and rutting data. Roughness and rutting are used as proxy for disaster consequences, for example, flooding, hurricane or cyclone.

Roughness and rutting parameters are vital for representing pavement structural and functional responses, and they are widely used. Roughness is a function of pavement structural and functional conditions and environmental factors. Roughness is developed due to cracking, rutting, potholes, depressions, ravelling, loading, pavement strength and drainage. It has been observed that roughness has a direct relationship with Vehicle Operating Costs (VOC), accidents and comfort (Odoki and Kerali, 2000; Prozzi, 2001). It shares relationships with design and explanatory variables, and inherent pavement deterioration. Therefore, roughness is the single most representative of pavement performance which has a positive co-relation with the Pavement Serviceability Index (a basic parameter in the AASHTO design method), and it is also widely used. It is related to the treatment trigger levels in developing a M&R strategy (Khan, 2006). On the other hand, rutting is also an important factor which relates to pavement structural strength, skid resistance and accidents. A pavement may fail at bituminous layer due to tensile strain and at subgrade due to compressive strain which links to rutting. Pavement rehabilitation design is related to deflection and rutting. Hence, a rutting based RD model with flooding is also considered.

VOC is related to vehicle and road characteristics, and sometimes known as 'Brazil relationships' (Archondo-Callao and Faiz, 1994). The Brazil study provided rational models for vehicle speed and fuel consumptions, and a better one for vehicle speed and road roughness (Watanatada, et al. 1987). The VOC models are mechanistic, and may be used in different countries. In general, the higher the road roughness, the higher is the VOC; and this value is changed due to different vehicle types and characteristics. The detailed VOC model and its sub-models, i.e., speed-flow, fuel consumption, congestion and parts consumption, are discussed in Odoki and Kerali (2000).

The percentage transition method uses real data to derive TPMs for representative road groups. It estimates transition from one state to another in the next year, and generates a TPM. This method can address explanatory variables and inherent pavement deterioration. The study uses the roughness and rutting distribution data to derive non-homogeneous TPMs with and without flooding, and then a Monte Carlo simulation for generating roughness and rutting-based RD models.

For developing RD models for a road group, IRI and rutting vs. time relationships are needed with the observed data. These trends will ensure how many TPMs are necessary to obtain a final TPM for simulation. Generally, a new TPM is generated when there is a change in deterioration trend. A final TPM is derived for the years with no flooding from several TPMs and one TPM is used to reflect a flood. The final two TPMs, that is, a TPM without flood and a TPM with flood, are used in the simulation.

The current simulation is undertaken in MATLAB. In each simulation, a set of random variables are used to compare with the flooding probability and to determine if a normal TPM or a flood TPM should be utilised. The chance of selecting a flood TPM depends on the chance of a flood occurrence. A second random variable is generated to estimate the future state of a road. The final TPM is averaged over all the simulated states. After all the simulations are completed, the RD models are generated for different probabilities of flooding. In the current RD modelling, this procedure is continued for 10,000 trials over a 20-year period.

The TMR-QLD has records of detailed road inventory, road conditions, traffic and pavement historical data for its road network. The 34,000 km road database covers roughness and rutting data for about 10-12 years. Road deterioration as a result of flooding (with IRI and rutting data) is also captured in the database. Data quality checking was done to get the reliable road condition data through correcting missing data and zero values and assessing deterioration trends. Pavement performance trends (IRI vs. time) and any IRI jumps were checked for a road group. Very few data were found missing which was checked using deterioration trends from similar types of roads. It is worth noting that only the available reliable data were used in the analysis. Therefore, it was possible to develop IRI and rutting deterioration trends and with and without flooding TPMs for each road group; and ultimately a RD model which reflects flooding.

The current analysis considered three types of pavement (flexible, rigid and composite), three categories of traffic loading (high, medium and low) and three types of pavement strength (high, moderate and low) in road groupings which gives 27 road groups for the network. It is worth noting that a composite pavement is a road that has stabilised granular layers either in the base or subbase or both. However, the granular layers are not completely concrete. Apart from that this road has asphalt as a surface layer.

After statistical analysis of 34,000 km road data, loading and strength ranges were set for the road groupings. The criteria set for the three types of loading were: low (<1 Million Equivalent Single Axle Loading in design life, MESAL), medium (1-10 MESAL) and high (>10 MESAL). The representative road groupings show that low traffic loading pass through all types of roads, which has been shown in Appendix. The three types of pavement strength were calculated from the available pavement age, seal age and pavement depth data. A relationship was assumed to obtain a hypothetical strength value, i.e., strength = (1/pavement age) * (1/seal age) * pavement depth. After obtaining these data, pavements were categorized into three groups, i.e., poor (<1), fair (1-5) and strong (>5). The new specific RD model is valid for a particular road group; and as a result, all of these RD models are suitable for the whole road network.

Generally, a strong road is related to pavement structural strength (SN), which is depended on different pavement layers thicknesses and layer coefficients/Mr. Pavement age and seal age effect on road deterioration, and these two factors along with pavement depth (based on different layers' thicknesses) are related to strength. Therefore, these three available parameters' data were used to get a strength value. The TMR-QLD's road database did not have detailed materials characteristics information. As a result, materials properties were not used in estimating strength values. Generally, an older pavement has lower strength value compared to a new one if other factors remain unchanged; similarly a newly sealed and/or a thicker pavement have higher strength values. On the other hand, a thicker pavement gives a higher strength. A pavement age reflects time passed after construction, while a seal age gives time passed after sealing of the surface layer is done for periodic maintenance.

The development process of RD models with flooding is summarised below. It is worth noting that no rutting model has been developed for the rigid pavements as they do not have rutting.

Inputs to the RD model development:

- Road groups;
- IRI and rutting vs. time data; and
- IRI and rutting distribution data.

Techniques to derive RD models:

- Percentage transition method for TPMs with and without flooding; and
- Monte Carlo simulation.

Outcomes of the RD modelling:

- IRI and rutting-based RD models for all road groups and also site-specific models.

Figure 3.2 shows the overall approach in deriving RD models for the twenty-seven road groups. The roughness and rutting vs. time are assessed to obtain trends on pavement performances for a road group. With and without flooding TPMs are generated from the change in trends. These TPMs are used in the Monte Carlo simulation to obtain roughness and rutting-based RD models for a road group. Moreover, ΔIRI and incremental change in rutting (ΔRutting) have been generated for different probabilities of flooding.

Figure 3.3 reveals the simulation process (code used) to obtain RD models for different flooding probabilities. As mentioned, random variables are generated to select either with or without flood TPMs. Then, another set of random variables are used to consider failure probability at a given state. After 10,000 trials for a 20-year period, an average RD model is obtained.

This approach was used for deriving the RD models with flooding both for representative and site specific roads. These results were verified with the actual data in Logan City of Queensland, which was observed after the January 2011 flood. Therefore, it is essential to know the details of the January 2011 flooding in Logan. To validate the new RD models, the study utilizes the flooding maps of Logan at one flood in five years, one flood in two years and one in every year which is respectively equivalent to 20%, 50% and 100% probability of flooding to identify the flooded roads. All the major roads under the TMR-QLD in Logan area are checked with the flood maps whether they were affected or not in the January 2011 flooding. The actual probability of this flood has also been determined using the flood records of the Logan River obtained by the Bureau of Meteorology (BoM). The short-term up to 2 years' roughness and rutting-based RD models for the affected roads were generated at that flooding probability, which has been checked with the representative RD models and real data captured for the flood affected roads. The model predictions and field data are also checked with a t-test. The step-wise approach is given in *Figure 3.4*.

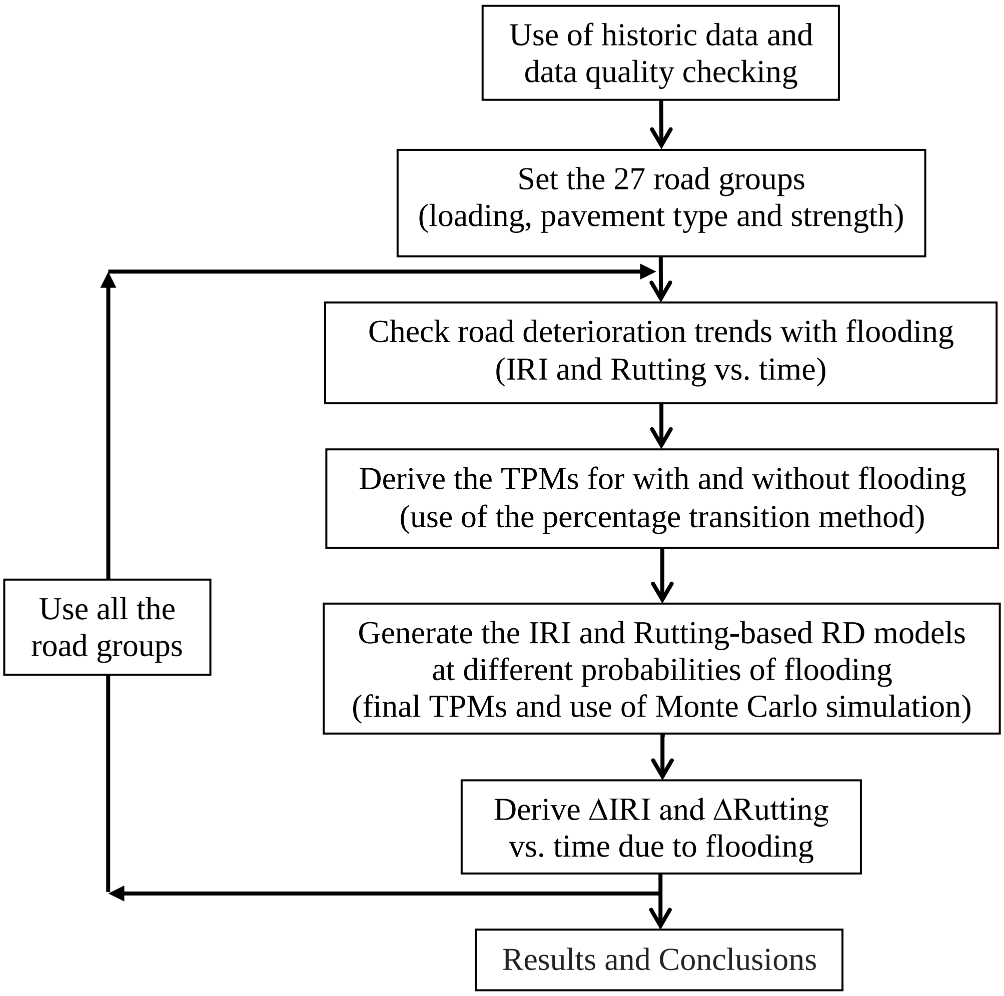

Figure 3.2: Approaches in deriving the RD models with flooding

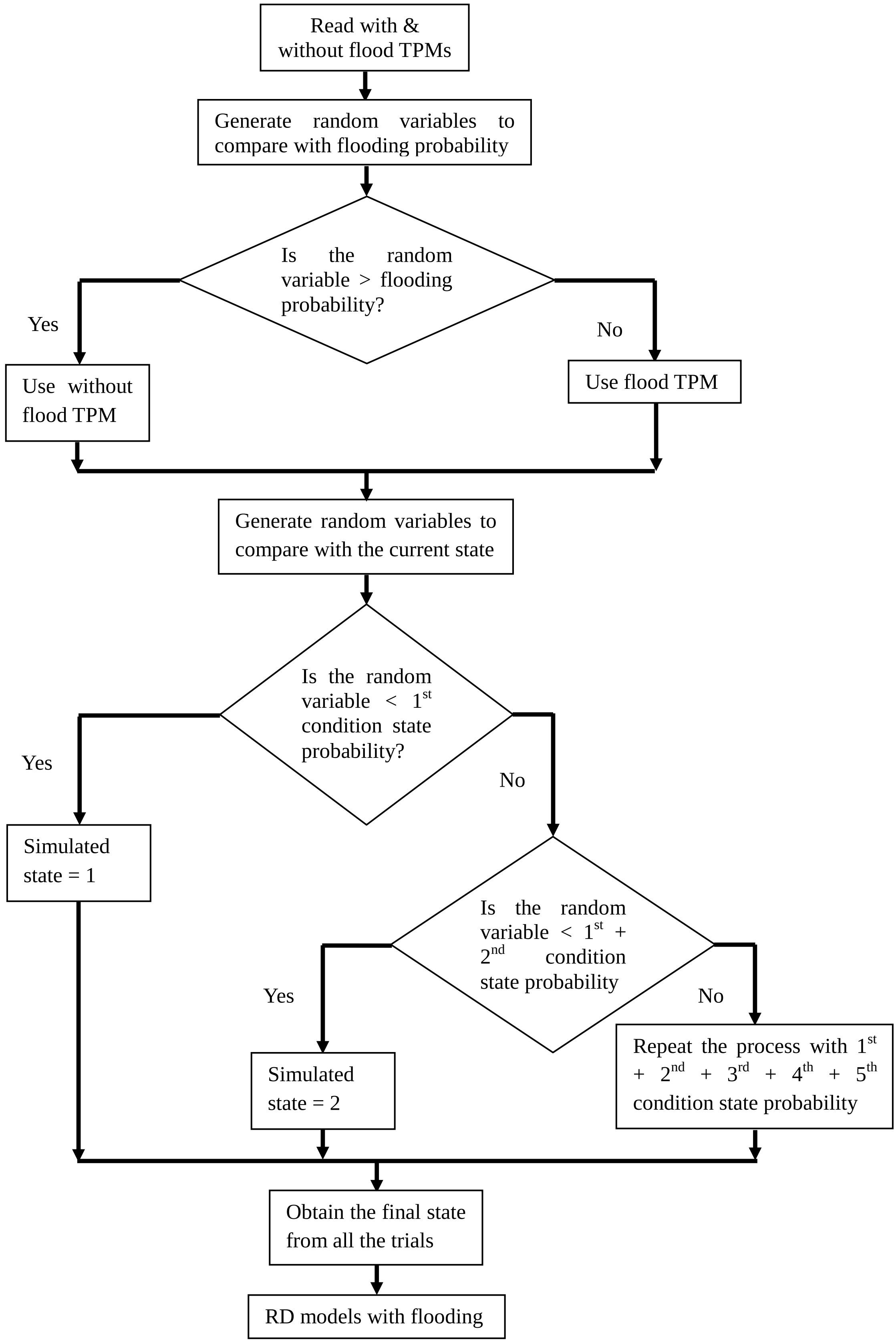

Figure 3.3: Simulation process to generate the RD models incorporating flooding

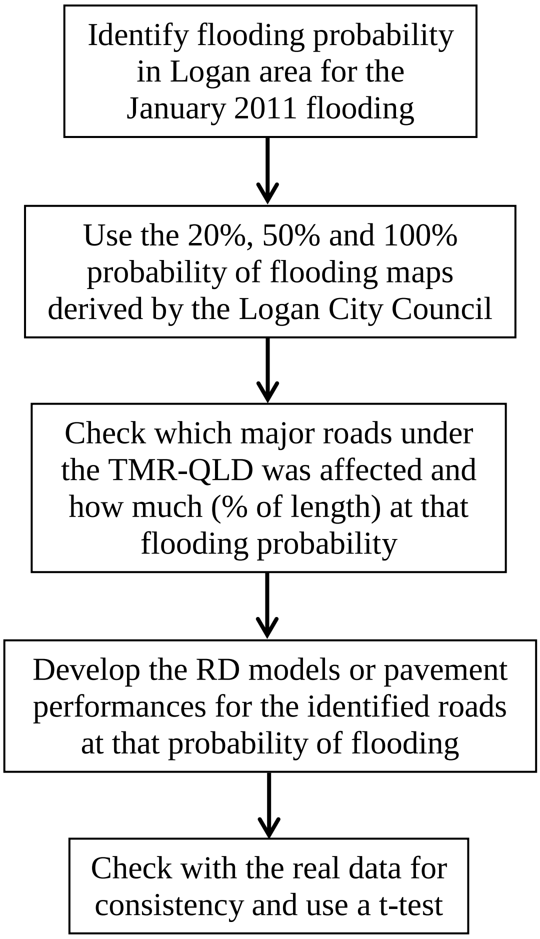

Figure 3.4: The approach proposed to validate the new RD models

3.2 Impact of Mr Loss on Pavement Performances

Flooding reveals moisture intrusion in a pavement which reduces Mr at the granular and subgrade layers, ultimately pavement strength is reduced. As a result, a pavement performance would be poorer. This study has used the EICM model of the AASHTO 2008 (AASHTO, 2008) to determine the Mr loss, which has been discussed in Section 2.6. Then, these Mr loss values have been used as inputs in the AASHTO 2008 and HDM-4 analysis for predicting pavement performances (ΔIRI in year 1) for the flexible pavement road groups in Queensland. Seven flexible pavements are chosen here as an example to show pavement performances due to Mr losses. In future, composite and rigid pavements may be considered. Obtaining flood-resilient pavement drives to develop a relationship between ΔIRI vs. Mr loss during flooding. The current approach is given in *Figure 3.5*.

In this study, different a, b and k_m values of the EICM model, suggested by Cary and Zapata (2010) and Zapata, et al. (2007) for the course and fine grained soils, are used to get the Mr loss due to different moisture intrusion scenarios.

Queensland soils are mainly grouped into several categories (QLD Govt., 2013), which may mostly be similar to the A-6 (clayey soil), A-2-6 (silty sand), A-5 (silty soil) and A-3 (fine sand) of the AASHTO classifications. Therefore, they have both coarse-grained and fine-grained materials as per the Zapata, et al. (2007). The laboratory testing was outside the scope, hence the suggested a, b and K_m values for coarse and fine-grained soils (Cary and Zapata, 2010; Zapata, et al. 2007) were used as inputs in the EICM model which provided Mr values for different layers due to moisture intrusion. The output results of the analysis were checked with the previous studies and found reliable; which has been discussed in Chapter Five. Therefore, the factors of Cary and Zapata (2010) and Zapata, et al. (2007) are reasonable. In future, these factors may be generated for the Australian condition.

The estimated varying Mr values have been used to predict the ∆IRI changes in year 1. As the HDM-4 model's roughness prediction is related to SN that is a function of Mr, therefore, the above varying Mr values are utilised to get SNs with the AASHTO 1993 method (AASHTO, 1993). These varying SN values due to Mr losses are used as inputs in the ∆IRI prediction using the HDM-4 model. Details of the HDM-4 roughness progression can be seen in Odoki and Kerali (2000).

Similarly, the roughness prediction model of the AASHTO 2008 is related to rutting that links to compressive strain at the top of subgrade (ε_c) and tensile strain at the bottom of asphalt layer (ε_t), and ultimately to Mr (AASHTO, 2008; Darter, et al. 2009; Papagiannakis, 2013). Therefore, the Method of Equivalent Thickness (MET), an analytic method of pavement design, is used to obtain ε_c and ε_t values at varying Mr for granular and subgrade layers, which have been utilised as inputs in the AASHTO 2008 model to get ∆IRI. Details of the MET can be seen elsewhere (Khan, 2005). It is worth noting that the updated IRI prediction model of AASHTO 2008 has been used in the analysis.

The layered Elastic theory, MET and Finite Element Method (FEM) are the most commonly used analytical models for pavement analysis. All analytical models involve some simplifications with respect to real conditions and have some advantages as well as disadvantages. Although the non-linearity of materials, spatial variations of modulus (change in modulus value with radial distance) and the dynamic loading of vehicles can be addressed by the FEM, it has some limitations in accurately analysing the pavement system and also it takes a large amount of computer time (Huang, 1993).

On the other hand, MET is simple, calculations can be done in a spreadsheet, and most importantly, it gives reasonably accurate results (Ullidtz, 2002; Ullidtz, et al. 2003). Therefore, the MET has been chosen. However, material non-linearity is not considered due to not having any data during extreme moisture intrusion/flood, which is also not in the scope. Two previous studies (Kim, et al. 2009 and Ghadimi, et al. 2013) revealed the difference of results in pavement design due to granular materials linearity and non-linearity. A non-linear analysis results provided about 4-6% higher deflection, 13-33% higher ε_t, and 0.4-4% higher ε_c. Generally, rutting failure due to ε_c is more critical during flooding/extreme moisture along with deflection, and their differences are not significant. Therefore, a pavement analysis with materials linearity was reasonable.

The MET is a pavement design method that depends on the Burmester's layer elastic theory and considers materials as linear elastic (Khan, 2005). In this analysis, three layers are assumed for a flexible pavement, i.e., surface, base and sub-base as granular layers and subgrade.

The AASHTO 2008 roughness prediction models results due to extreme moisture intrusion at granular and subgrade layers are indicative because of different assumptions of the factors, while HDM-4 one provides reasonable results. However, these findings are checked with the actual data obtained after the January 2011 flood for two major roads in Logan, Queensland. It ensures consistency of the ΔIRI predictions in year 1, which are used for obtaining flood resilient pavements and pavement life-cycle performances due to Mr loss at year 1. These results also show reliability of the new RD models.

The AASHTO 2008 roughness prediction model (AASHTO 2008 and Darter, et al. 2009) for asphalt pavements is given below, which has been used here.

$$IRI = IRI_o + 0.0150\,(SF) + 0.400\,(FC_{Total}) + 0.0080\,(TC) + 40.0\,(RD) \qquad (3.1)$$

where,

IRI_o = Initial IRI (in/mile) after construction;

SF = Site factor;

FC_{Total} = Area of fatigue cracking (combined alligator, longitudinal, and reflection cracking in the wheel path), percent of total lane area. All load related cracks are combined on an area basis – length of cracks is multiplied by 1 foot to convert length into an area basis;
TC = Length of transverse cracking as ft/mile (including the reflection of transverse cracks in existing hot mixed asphalt pavements); and
RD = Average rut depth (in).

The SF may be estimated using the following equation.

$$SF = FROSTH + SWELLP * AGE^{1.5} \tag{3.2}$$

where,
FROSTH = Ln ([PRECIP+1] * FINES * [FI+1])
SWELLP = Ln ([PRECIP+1] * CLAY * [PI+1])
FINES = FSAND + SILT
AGE = pavement age (years);
PI = subgrade soil plasticity index;
PRECIP = mean annual precipitation (in);
FI = mean annual freezing index (deg. F Days);
FSAND = amount of fine sand particles in subgrade (percent of particles between 0.074 and 0.42 mm);
SILT = amount of silt particles in subgrade (percent of particles between 0.074 and 0.002 mm); and
CLAY = amount of clay size particles in subgrade (percent of particles less than 0.002 mm).

The basic HDM-4 roughness model (Odoki and Kerali, 2000) which has been used in the analysis is shown below. The models for estimating yearly incremental roughness changes due to structural deterioration, cracking, rutting, potholing and environment are not shown here, which can be seen in Odoki and Kerali (2000).

$$\Delta IRI = K_{gp} [\Delta IRI_s + \Delta IRI_c + \Delta IRI_r + \Delta IRI_t] + \Delta IRI_e \tag{3.3}$$

where,
ΔIRI = total incremental change in roughness during the analysis year (IRI, m/km);
K_{gp} = calibration factor for roughness progression;

ΔIRI_s = incremental change in roughness due to structural deterioration during the analysis year (IRI, m/km);
ΔIRI_c = incremental change in roughness due to cracking during the analysis year (IRI, m/km);
ΔIRI_r = incremental change in roughness due to rutting during the analysis year (IRI, m/km);
ΔIRI_t = incremental change in roughness due to potholing during the analysis year (IRI, m/km); and
ΔIRI_e = incremental change in roughness due to environment during the analysis year (IRI, m/km).

The analyses assumed many factors and in some cases local calibration factors are used (Cary and Zapata, 2010; Zapata, et al. 2007; Austroads, 2010; TMR, 2013; AASHTO, 2008; Rashid, et al. 2013; TMR, 2010), which are shown in *Table 3.1*. However, the results are checked at different stages for reliability. Although, the findings are indicative, they matched closely with the actual field data. It should be noted that an accurately calibrated model with extensive local data should be used for precise prediction purposes. Nonetheless, the proposed analysis is an indicator to demonstrate the relative deterioration of pavement performances due to flooding compared to the common practise of ignoring flood events.

Table 3.1: Sources of the major calibration factors of the AASHTO 2008 and HDM-4 roughness models

Models Used	Key Calibration Factors	Value	Sources
The EICM model	Maximum modulus ratio	Coarse-grained soil: 2.0 Fine-grained soil: 2.5	Cary and Zapata, 2010 and Zapata, et al. 2007
	a = minimum of log (Mr/Mropt)	Coarse-grained soil: -0.3123 Fine-grained soil: -0.5934	Cary and Zapata, 2010 and Zapata, et al. 2007
	b = maximum of log (Mr/Mropt)	Coarse-grained soil: 0.3 Fine-grained soil: 0.4	Cary and Zapata, 2010 and Zapata, et al. 2007
	km = regression parameter	Coarse-grained soil: 6.8157 Fine-grained soil: 6.1324	Cary and Zapata, 2010 and Zapata, et al. 2007
The MET model	Subgrade California Bearing Ratio	5%	Austroads, 2010 and TMR, 2013
	Subgrade modulus in vertical direction	50 MPa	Austroads, 2010 and TMR, 2013
	Base modulus in vertical direction	500 MPa	Austroads, 2010 and TMR, 2013
	Sub-base modulus in vertical direction	125 MPa	Austroads, 2010 and TMR, 2013
	Tire pressure	0.75 MPa	Austroads, 2010 and TMR, 2013
	Tire distance	92.1 mm	Austroads, 2010 and TMR, 2013
AASHTO 2008	*Equations and Factors on:* Fatigue failure Fatigue cracking Transverse cracking Rutting IRI prediction	Default values, materials data and existing road condition	AASHTO, 2008
HDM-4	Calibration factor for roughness progression	Sealed roads: 0.90 Asphalt roads: 0.55 Concrete roads: 0.33	Rashid, et al. 2013 and TMR, 2010
	Calibration factor for roughness progression due to environment	1.0	Rashid, et al. 2013 and TMR, 2010
	Drainage factor	1.0	Rashid, et al. 2013 and TMR, 2010

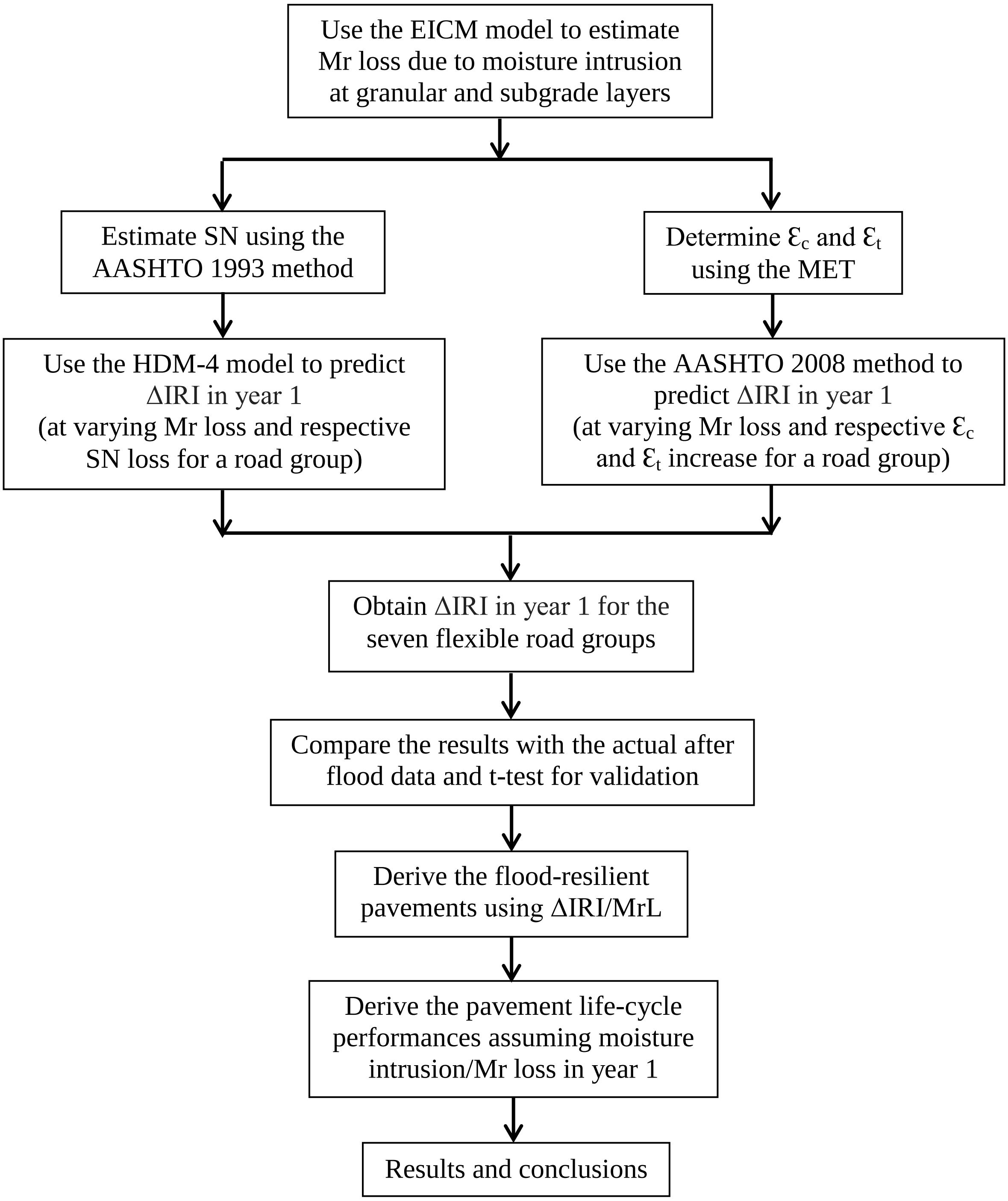

Figure 3.5: Approach to assess pavement performance at varying Mr due to moisture at untreated layers

3.3 Development of the Flood-resilient Pavements

The study aims to obtain the flood-resilient pavements using the findings from i) the new RD models, ii) impact of Mr loss on pavement performances, and iii) flood risk analysis results. The roughness and rutting-based RD models were used to assess different pavement performances with flooding. In addition, a new gradient of ΔIRI/Pr was considered to obtain the impact of flooding on a pavement's performance. The current research used the Mr loss values as inputs in the AASHTO 2008 and HDM-4 roughness models for predicting pavement responses, that is, ΔIRI in year 1. Obtaining flood-resilient pavement generates a new ΔIRI/MrL which helps to visualise pavement performances due to a flood.

A detailed review has been done using the current research results from i) different pavement performances with flooding, ii) ΔIRI/Pr, iii) ΔIRI/MrL, and iv) flood risk and consequence scores. All these provide a clear picture on pavements' flood resilience. *Figure 3.6* shows the approach in obtaining a flood-resilient pavement.

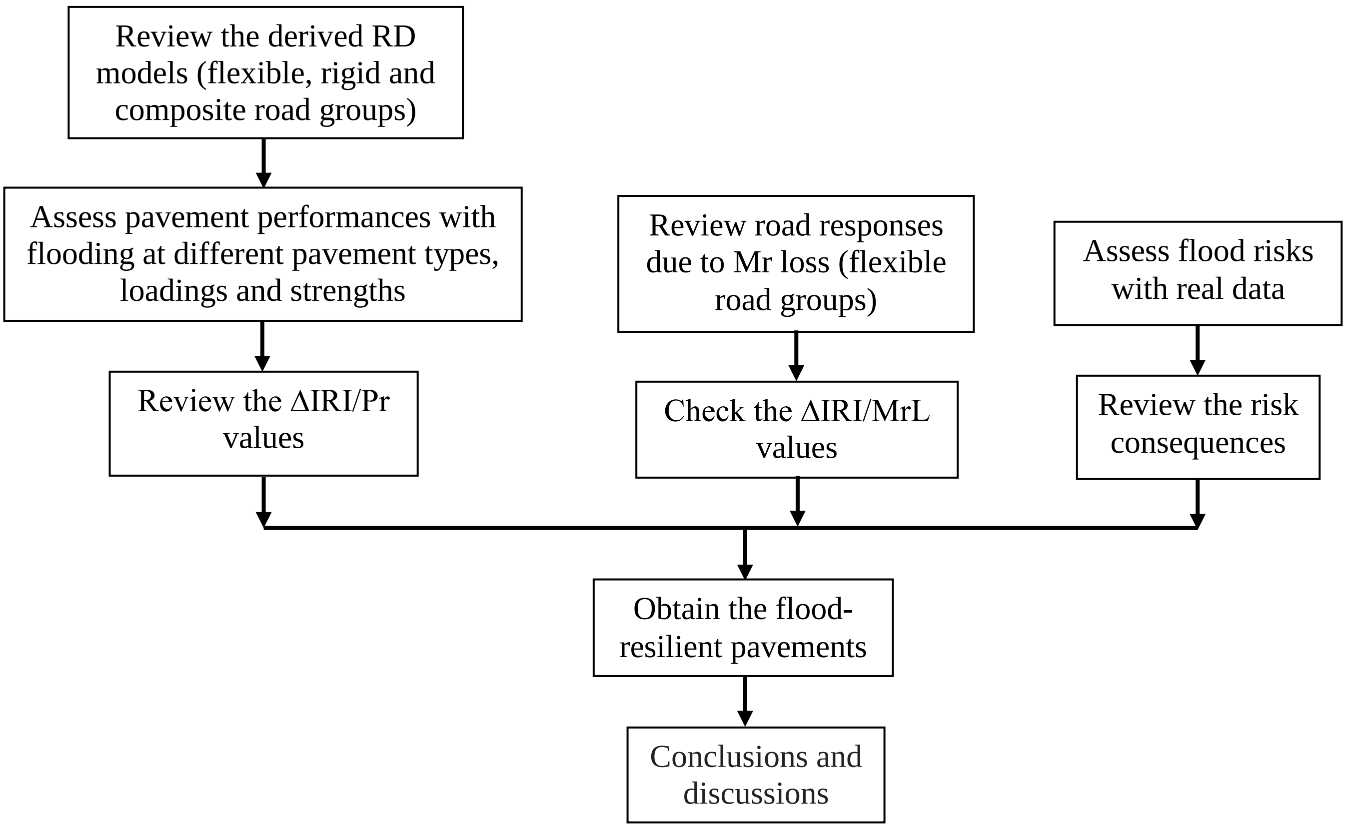

Figure 3.6: Approach to obtain a flood resilient pavement

3.4 Derivation of Optimum Maintenance Standards and Strategies for TMR-QLD

The HDM-4 strategy analysis was considered to derive optimum maintenance standards and strategies. The optimisation algorithm has been shown in Chapter Two. Each of the 27 road groups was assigned with 6 standards and 6 treatments, i.e., 36 standard-treatment alternatives (M_s as discussed in the optimisation algorithms in Equation 2.11 to 2.13) as input in the HDM-4 analysis.

The TMR-QLD uses 5.5 IRI (m/km) as the ultimate roughness value; therefore, six maintenance standards, i.e., 3.0, 3.5, 4.0, 4.5, 5.0 and 5.5 IRI were used as standards to maintain a road. The authority's current treatment alternatives were used for each standard, i.e., six routine and periodic maintenance treatments. As a result, Routine Maintenance (RM), seal coat, slurry seal, OV 30 mm, OV 45 mm and OV 75 mm were used as treatment alternatives. The HDM-4 strategy and optimization analysis results provide a cost-effective standard with a recommended treatment for each road group. The necessary budget to keep a road at a set standard was also developed. The optimization objective considered was "minimise agency cost and maximise performance".

A similar approach was used for obtaining optimum maintenance standards and strategies for the whole road network in Bangladesh (Khan, 2006; Khan and Odoki, 2010). The inputs and optimisation objectives along with the probable outcomes are given below. The optimisation algorithm given in Section 2.8 shows that agency cost, pavement performance and economic benefits are the criteria for optimisation, while budget is used as a constraint. It is ensured that an agency cost does not exceed the budget. The vehicle costs data for different categories of vehicles are used as inputs in estimating VOC, ultimately benefits. The ΔIRI reduction is used to evaluate pavement performances. The lower the ΔIRI change in a year, the better is the performance.

Inputs for Optimisation:

- Developed RD models;
- Road condition data;
- Traffic data;
- Pavement history data;
- Road geometry data;
- Vehicle costs data;
- Treatments with criteria and costs;
- Proposed standards (3.0 IRI, 3.5 IRI, 4.0 IRI, 4.5 IRI, 5.0 IRI, 5.5 IRI); and
- RM, seal coat, slurry seal, OV 30 mm, OV 45 mm and OV 75 mm.

Optimisation Objectives (at budget constraints):

- Minimise agency costs; and
- Maximise performance target.

Outputs of optimisation:

- Optimum pavement maintenance standards;
- Optimum M&R strategy; and
- Necessary budget.

The overall methodology to develop optimal pavement maintenance standards and strategies at without flood condition can be seen in *Figure 3.7*. The HDM-4 outputs especially economic results and pavement performances were assessed for the 36 standard-treatment alternatives. The Net Present Value (NPV), NPV/Cost and Internal Rate of Return (IRR in %) were compared to obtain the optimum standard and necessary treatments for a road.

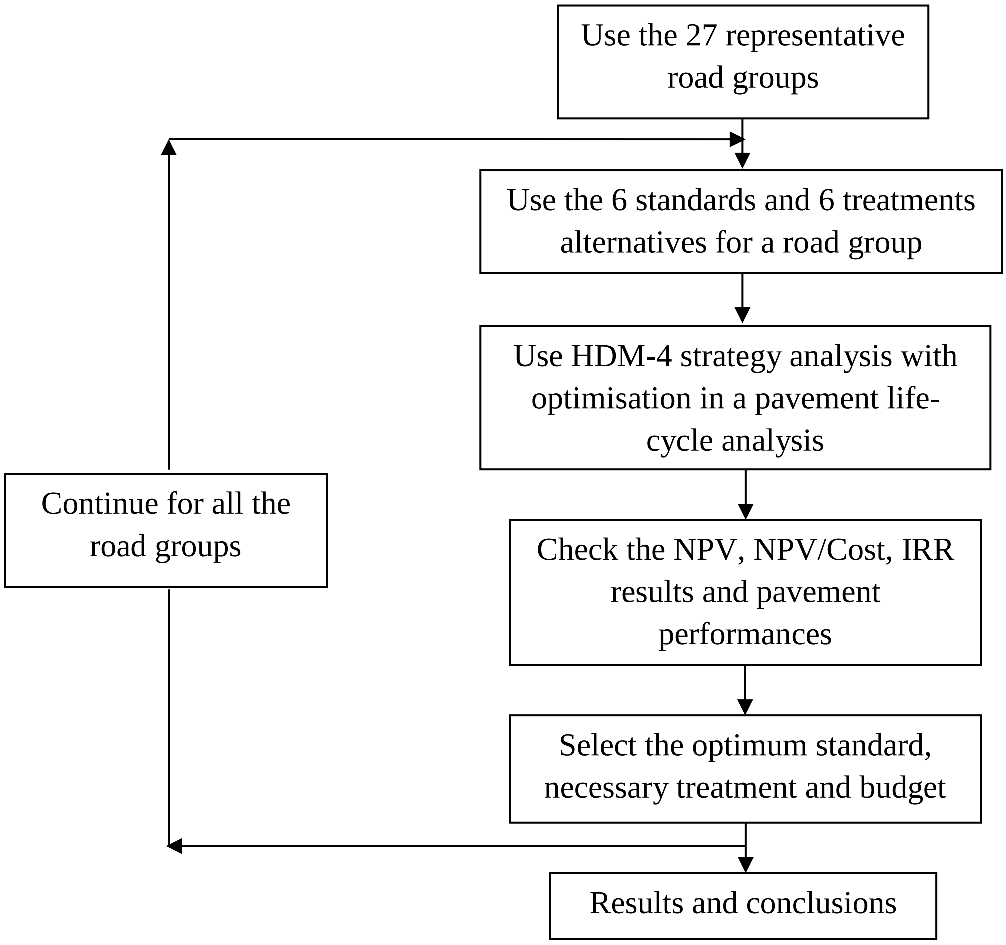

Figure 3.7: Proposed methodology to derive optimum maintenance standards

3.5 Development of a Post-flood Road Maintenance Strategy

A post-flood strategy reveals a pavement life-cycle performance with a flood, and recommends for necessary treatments as a post flood rehabilitation. This analysis assumes that asphalt overlay and/or granular and subgrade layers' stabilization could be used as a post-flood treatment. Stabilisation at granular and subgrade layers is done with cement/lime/bitumen and at surface layer with foamed bitumen. It needs removal of the existing top layers while stabilising.

For deriving a post-flood strategy, a flood has been assumed in year 1 and treatments were given starting from year 2. Three different years were chosen here, as it was not easy to rehabilitate the whole network given the time required to design post-flood rehabilitation, procurement process and allocation of funding; which was also the case for the TMR-QLD.

The current analysis is unique, as it uses the newly derived roughness and rutting-based RD models with flooding in year 2, 3 or 4 for deterioration prediction before post flood rehabilitation were given. The study uses 6 maintenance standards: 3.0, 3.5, 4.0, 4.5, 5.0 and 5.5 IRI, and 13 treatments for rehabilitation. Therefore, a total of 78 standard-treatment alternatives were used to get the optimum one. The currently practiced seven treatments by TMR-QLD were considered along with the six newly proposed treatments for rehabilitation: OV 100 mm, OV 150 mm, OV 200 mm, stabilisation + OV 100 mm, stabilisation + OV 150 mm, and stabilisation + OV 200 mm. The current thesis has assumed new pavement strength and base modulus values after a stabilisation with strengthening overlay treatment in the HDM-4 analysis. After post-flood rehabilitation, a normal pavement deterioration was assumed in the remaining life-cycle analysis. Therefore, only one flood event was used in a life-cycle.

The current post-flood maintenance strategy development framework is valid even if two or more floods come in life-cycle. For each flood case, the RD models would be used to predict pavement deterioration for two to three years after a flood, and rehabilitation would start from the next to the flood year. This would provide a post-flood strategy for each specific flood.

The current study uses the HDM-4 model for developing a post-flood maintenance strategy considering pavement LCA. The model uses three optimisation objectives: i) maximise economic benefits measured in NPV; ii) minimise ΔIRI; and iii) minimise agency cost at target IRI. The results could provide solutions considering both constrained and unconstrained budgets.

The *Figure 3.8* reveals the detailed approach used in this study. However, as an example, if a flood comes in year 10, then additional maintenance costs with no flood condition are needed between year 1 and 9. It has been found that about $0.9bn is needed in a year with no flood scenario. As a result, additional $8.1bn is needed for maintaining roads between year 1 and 9. Furthermore, if two or more flood events are occurred in life-cycle, then additional post-flood rehabilitation work is needed with extra costs. Road deterioration with a specific probability of flooding needs to be estimated with the RD models with flooding, and then a HDM-4 analysis could provide additional costs for necessary treatments.

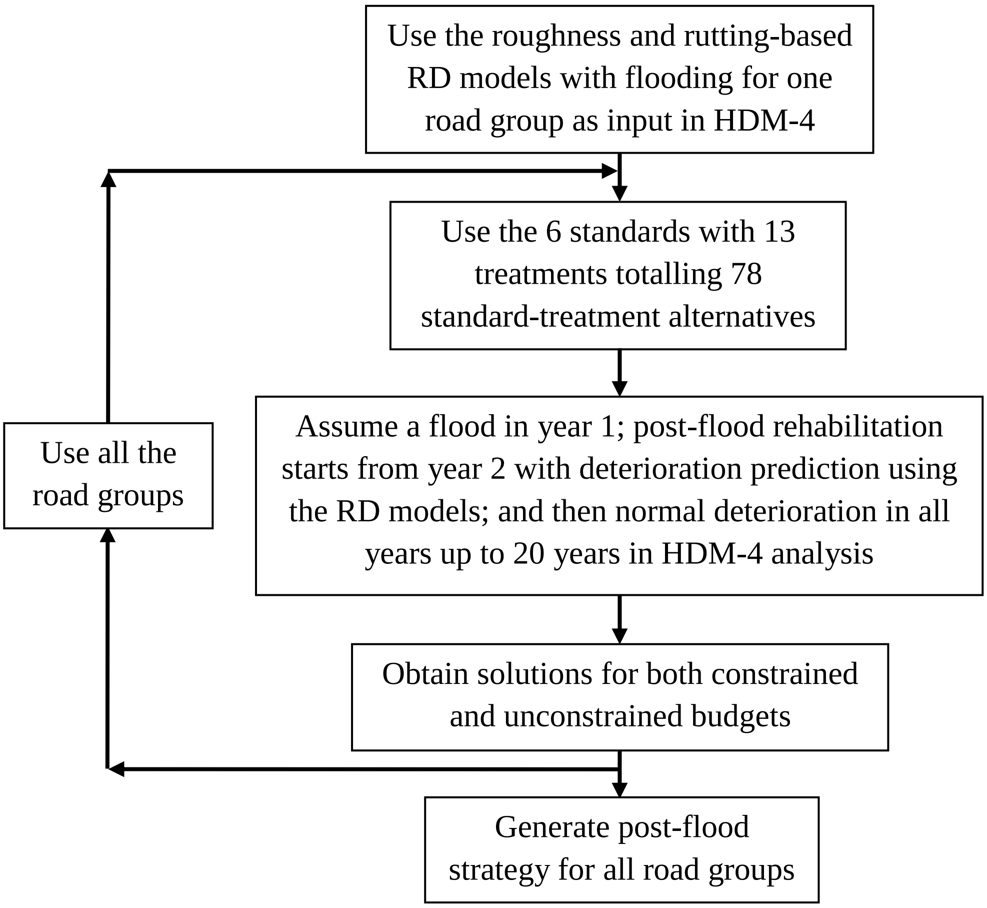

Figure 3.8: Approach in deriving a post-flood maintenance strategy

3.6 Development of a Pre-flood Road Maintenance Strategy

The current research derives a pre-flood road maintenance strategy using the HDM-4 model where strengthening treatments were given in year 1 and a flood was assumed randomly in year 2, 4, 7, 12 or 17. A strengthening treatment enhances the pavement structural strength, which may be an overlay and/or stabilization. If a road is constructed or treated in year 0 (before the analysis year), then no pre-flood treatment is needed, as this road is already at strong strength. A normal deterioration is presumed before a flood comes. The after flood deterioration for at least two years were predicted using the new roughness and rutting-based RD models. A two year window is required for a road authority to evaluate and implement post-flood treatments based on the Queensland experience (TMR, 2012b). For example, if a flood comes in year 7, the normal deterioration is predicted from year 2 to 7; and the RD models are used to predict deterioration in year 8 and 9. Then, the after flood rehabilitation was expected to be started from year 8 for some roads, as HDM-4 may suggest optimum treatments in this year. The remaining life-cycle period was assumed with normal deterioration.

There may be more floods, which have not been considered in the current scope of analysis. Moreover, the current approach/framework could be used for two or more flood cases. The analysis approach using one or more floods is highlighted below, although the current study has used one flood in life-cycle for simplicity in the analysis. As a result, it follows up to steps VI shown below. It clearly shows that a normal deterioration was predicted using the HDM-4 from year 2 until a flood comes. Then, after the first flood event, the roughness and rutting based RD models with flooding were used to predict road condition for the next two years for selecting appropriate post-flood treatments. Finally, normal road deterioration was then considered for the remaining period.

I. Provide pre-flood treatments in year 1;
II. Use a normal deterioration with HDM-4 and an optimum maintenance strategy until a flood comes;
III. Assume the first flood in any year (for example, at year 2, 4, 7, 12 or 17);
IV. Use the roughness and rutting based RD models with flooding for the first two years after the first flood;
V. Use post-flood treatments starting from the first year after the first flood;
VI. Use a normal deterioration with HDM-4 and an optimum maintenance strategy until the second flood comes;
VII. Use the second flood in any year;
VIII. Use the roughness and rutting based RD models with flooding for the first two years after the second flood;
IX. Use post-flood treatments starting from the first year after the second flood;

X. Use a normal deterioration with HDM-4 and an optimum maintenance strategy until the third flood comes; and

XI. Continue the process.

It was estimated that the treatments given in year 1 with the thin overlay recommended in the optimum strategy at no flood condition would suffice to keep all the roads at a desirable strength. Similar to the post-flood strategy analysis, this one also considered the same 6 maintenance standards and 13 treatments for rehabilitation, totalling 78 standard-treatment alternatives to get the optimum post-flood solution.

The pavements also perform better in normal condition as a result of this investment. The analysis assumed only one flood in a life-cycle and suggested necessary treatments and budget after that flood. As a result, necessary treatments and budget were obtained for year 1 and any year for after flood rehabilitation which was also included in this strategy to manage roads in its life-cycle. The results provide life-cycle pavement performances, necessary treatments at year 1 and any year after a flood, and the required budget. *Figure 3.9* reveals the detailed approach used in this study. All these results were compared with the derived post-flood strategy in Chapter Nine.

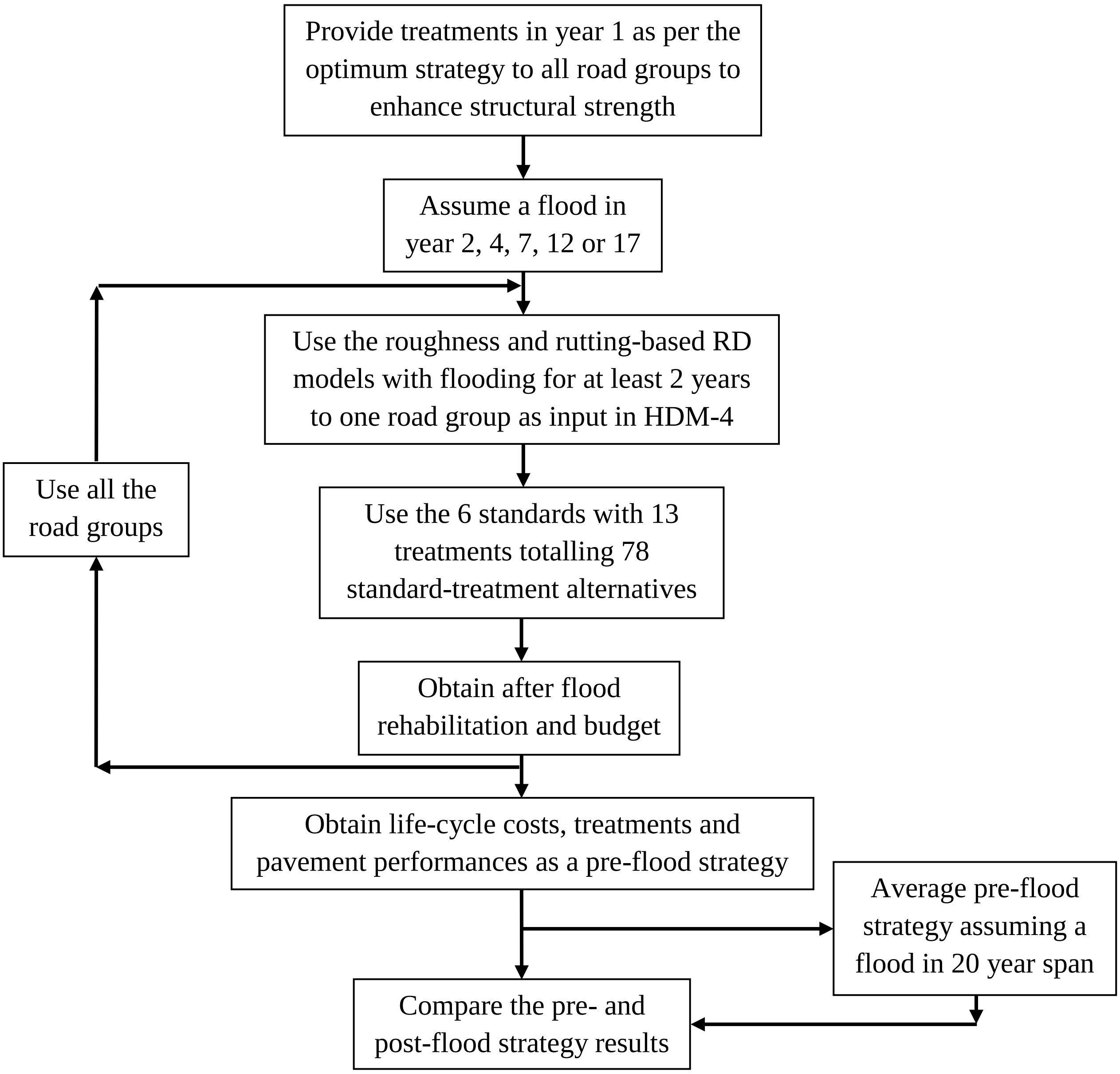

Figure 3.9: Approach in deriving a pre-flood maintenance strategy

3.7 Flood Risk Assessment Framework

A risk score is estimated using the relationship shown in Equation 2.14. In the current study, likelihood is considered using return period/frequency of a flood event and consequences are estimated from the impact of a flood on pavement performances presented with IRI. The roughness distribution data were used to get weighted average IRI values before and after a flood; and the IRI value before a flood was deducted from the IRI value after a flood to get a consequence score. While analysing IRI vs. time data, a flood is assumed when an abrupt jump in IRI in any year was observed. It is worth noting that there were not detailed recording of after flood data in the TMR-QLD database. Queensland is a flood prone area, and observes flash flooding, moisture intrusion and saturated conditions frequently in the low lying roads. In addition, a sudden jump in IRI was observed after a flood while examining the IRI data to derive the RD models with a flood. It is rare to get such an abrupt jump in IRI apart from a disaster like flooding. A combination of over loading and poor sub-surface drainage may effect on roughness jump, but the TMR-QLD's database did not have any information of these two factors. Therefore, an abrupt jump in IRI was used as an indication of a flood.

Both likelihood and consequences have been categorized into five groups as below. The likelihood was grouped based on engineering judgment. In the likelihood groupings, a 'rare' flood means a very low frequency of flood (1 in 100 years); and hence, it has been assigned the lowest weight of 1.

Likelihood of a flood event:

- Almost Frequent = 1 in 2 years (weight = 5)
- Likely = 1 in 5 years (weight = 4)
- Moderate = 1 in 10 years (weight = 3)
- Unlikely = 1 in 50 years (weight = 2)
- Rare = 1 in 100 years (weight = 1)

Similarly, engineering judgment was utilised for grouping of flooding consequences on pavement performances. Roughness increase value extracted from before and after a flood's roughness distribution data has been used for this purpose, as shown below. If a roughness increase value (ΔIRI) after a flood is less than 0.25 IRI, then it is termed as 'negligible'. On the other hand, an increase of greater than 2.00 IRI indicates 'extreme' consequence. Treatment costs could be considered as a factor for consequence, but because of not having sufficient information, treatment costs have been omitted. In addition, treatments are linked to the IRI changes, which are considered as consequence in this study.

Consequences:

- Negligible = < 0.25 IRI (weight = 1)
- Low = 0.25-0.50 IRI (weight = 2)
- Medium = 0.50-1.00 IRI (weight = 3)
- High = 1.00-2.00 IRI (weight = 4)
- Extreme = > 2.00 IRI (weight = 5)

As mentioned above, the consequence grouping is set using the pavement performances before and after a flood. The weights used for likelihood are based on frequency of a flood, commonly observed in the database. As both likelihood and consequence have 5 groups, the risk score is ranged between 1 and 25.

A flood risk matrix using *Equation 2.12* has been generated using the above likelihood and consequences, which is given in *Table 3.2*. The 'Black' area means regular flooding events with high consequences, and risk scores are higher than 12. A road authority must do the necessary measures to its road network in these areas. Similarly, the 'White' area means very rare flooding with less effect (score: 1 to 4), which does not need attention. However, the 'Gray' area needs appropriate attention, and it is considered a critical/moderate risk area with scores in between 4 and 12.

Risk scores and zoning:

- Low = score 1-4 ('White' zone)
- Moderate = score 4-12 ('Gray' zone)
- High = score 13-25 ('Black' zone)

The current study uses the IRI vs. time data between 2000 and 2012 for a road group, where an abrupt jump in roughness was assumed to be a result of a flood. The time gap between the two flooding years gives the frequency of a flood, i.e., likelihood; but 50 or 100 year likelihood could not be observed. The IRI values estimated from the roughness distribution results before and after a flood provide flooding consequence scores (see *Equation 3.3*). Then, the flood risk score is calculated for a road group using the likelihood and consequences (see *Equation 3.4*), which is used to obtain the flood-resilient pavements.

Consequence of a Flood = Average IRI after a Flood – Average IRI before the same flood (3.4)

Flood Risk = Likelihood of a flood * Consequence of the same flood (3.5)

The following four steps were followed in the current flood risk analysis:

- ➢ Use an abrupt roughness jump from the IRI vs. time data to identify a flood event;

- Assume likelihood of a flood from the time gap between the two floods, i.e., time between the current flood and the previous one;
- Obtain the consequence score from the IRI distribution results. Consequences of a flood were calculated using the *Equation 3.4*, which is the difference of average IRI values obtained from after and before a flood;
- Use the *Equation 3.5* to calculate the flood risk scores/rating and zoning; and
- Repeat the steps for all the road groups.

The results were used for obtaining a flood-resilient pavement.

Table 3.2: Flood risk matrix

Likelihood of a Flood Event	**Consequences**				
	Negligible (1)	**Low (2)**	**Medium (3)**	**High (4)**	**Extreme (5)**
Almost Frequent (5)	5	10	15	20	25
Likely (4)	4	8	12	16	20
Moderate (3)	3	6	9	12	15
Unlikely (2)	2	4	6	8	10
Rare (1)	1	2	3	4	5

3.8 Summary

This chapter discusses the detailed approaches used in the study. The overall research approach reveals that the new RD models with flooding are the vital component. These site-specific and network level models are validated and verified with the actual data and a t-test, which results are discussed in the next chapter. The models have been used for: i) developing optimum, pre- and post-flood strategies, ii) obtaining flood resilient pavements using the two new indicators and RD modelling results, and iii) flood risk assessment. Detailed methodologies along with flow charts for all of the above are highlighted separately.

CHAPTER FOUR: NEW RD MODELS WITH FLOODING[2]

4.1 Introduction

The current thesis focuses on deriving probabilistic IRI and rutting-based RD models that can address flooding using non-homogeneous TPMs as they can reflect pavement performances with uncertainty. The project and network level RD models are derived, which are validated with the actual data and t-tests. In the current research, the percentage transition method has been considered as it could help generate several TPMs based on pavement deterioration trends with IRI and rutting vs. time data. The method has used IRI and rutting distribution to derive non-homogeneous TPMs and a Monte Carlo simulation for RD results. The 34,000 km road data of TMR-QLD has been used that has after flood road condition data acquired in the database. In the analysis, roughness and rutting are used as proxy for a flooding. Key components discussed in this chapter are shown in *Figure 3.1*.

4.2 Major Inputs and Outcomes

The overall approach of the RD modelling is shown in *Figures 3.2 to 3.4.* The major inputs, techniques and expected outcomes in deriving the RD models are highlighted in Section 3.1 and given below.

Inputs to the RD model development:

- Representative road groups (see Section 3.1 for 27 road groups, which were based on 3 types of traffic loading: high, medium and low; 3 categories of pavement strength: high, moderate and low; and 3 types of pavement: flexible, composite and rigid) and some actual roads in Logan, Queensland;
- IRI and rutting vs. time data; and
- IRI and rutting distribution data.

Techniques to derive RD models:

- Percentage transition method for flooding and without flood TPMs; and
- Monte Carlo simulation (the simulation logic is shown in *Appendix 4A*).

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[2] *The findings provided in this chapter has resulted a Journal Paper:* http://www.icevirtuallibrary.com/content/article/10.1680/tran.13.00095

Outcomes of the RD modelling:

- With and without flood TPMs; and
- IRI and rutting-based RD models for the 27 road groups and some actual roads.

4.3 Indicative Results

At first, an indicative result on the trend of a RD model is shown here. The expected results relates to a roughness-based RD model for a hypothetical road group is given in *Figure 4.1*. It reveals that the new RD model can estimate the impact of a flood by providing IRI change with a jump after flooding, that is currently absent in a PMS. This is vital to select appropriate treatment for a post-flood rehabilitation.

As an example, a road deterioration envelop with 0% and 100% probability of flooding for the Rigid pavement, Low Traffic loading and Strong strength (R_LT_S) road group has been generated (see *Figure 4.2*). This new model uses a non-homogenous TPM and Monte Carlo simulation. *Figure 4.2* shows the road deterioration envelop with and without flooding. The final RD models with different probabilities of flooding are within this envelop. This result reveals that a pavement performance is affected in the first couple of years because of a flood in year 1.

Figure 4.3 provides an indicative life-cycle performance if a flood occurs in this period. Assuming 3.0 IRI as set standard, RM + PM are needed before a flood. A post-flood rehabilitation is needed after the flood, and again RM + PM would require for the remaining period to keep the road at set target. Therefore, $X million is needed before a flood, $Y million for a post-flood rehabilitation and $Z million for the remaining period. In this study, the HDM-4 model has been used to get pavement life-cycle performances and budget incorporating a flood.

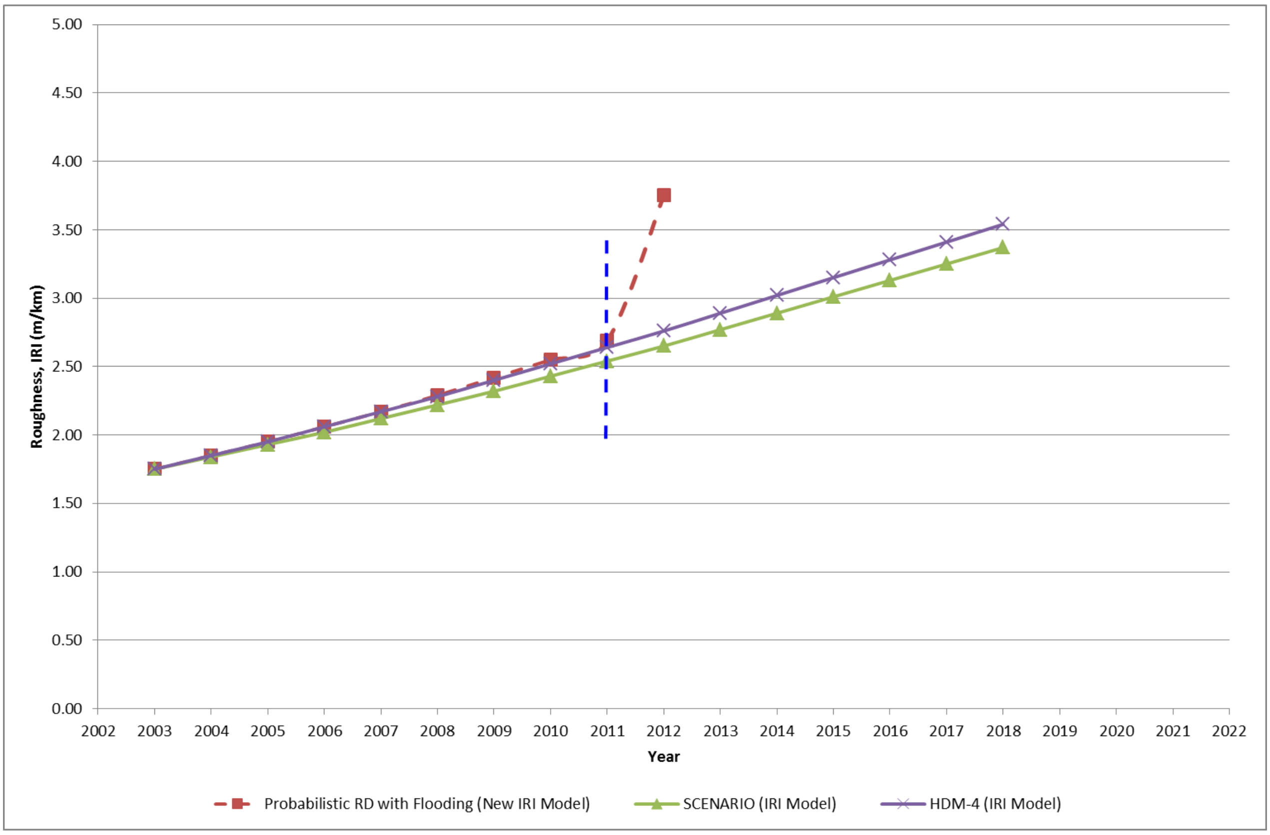


Figure 4.1: Comparison of the derived RD model with SCENARIO and HDM-4

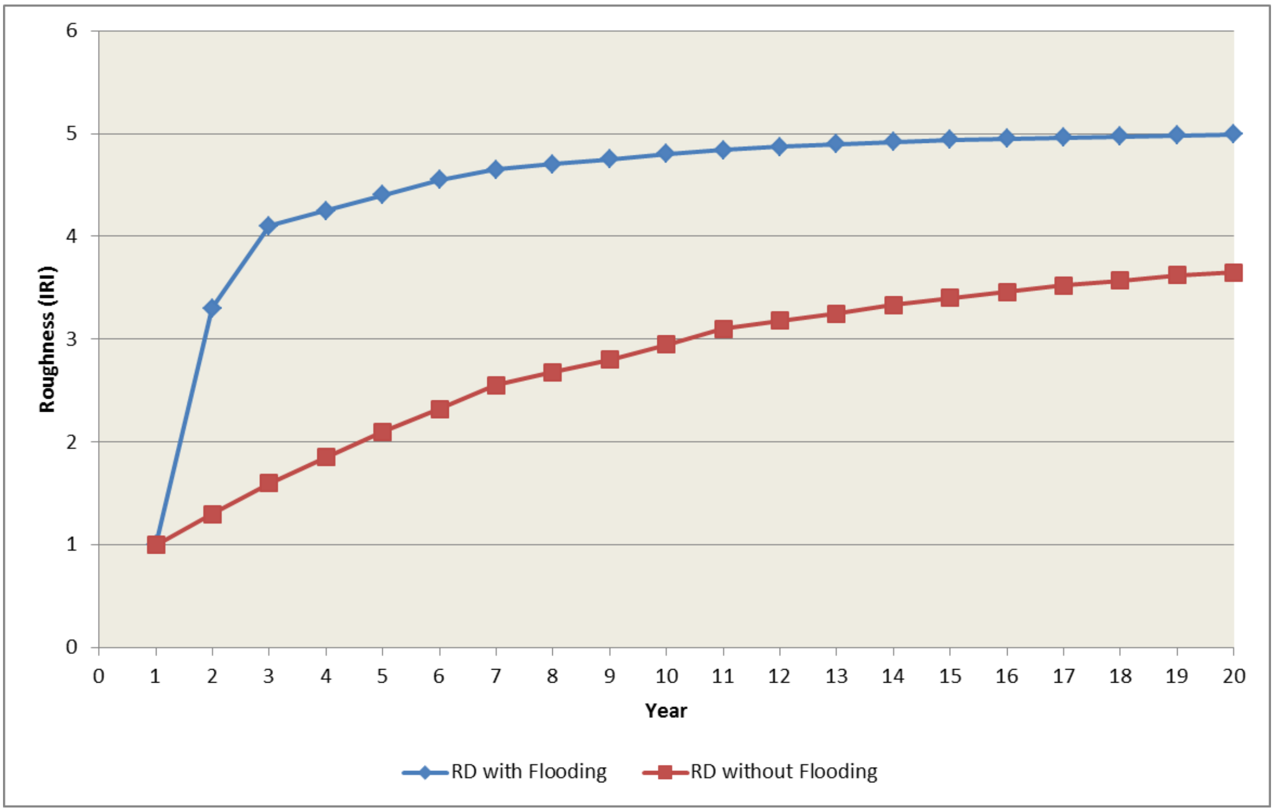


Figure 4.2: Proposed RD model with flooding for the R_LT_S road group

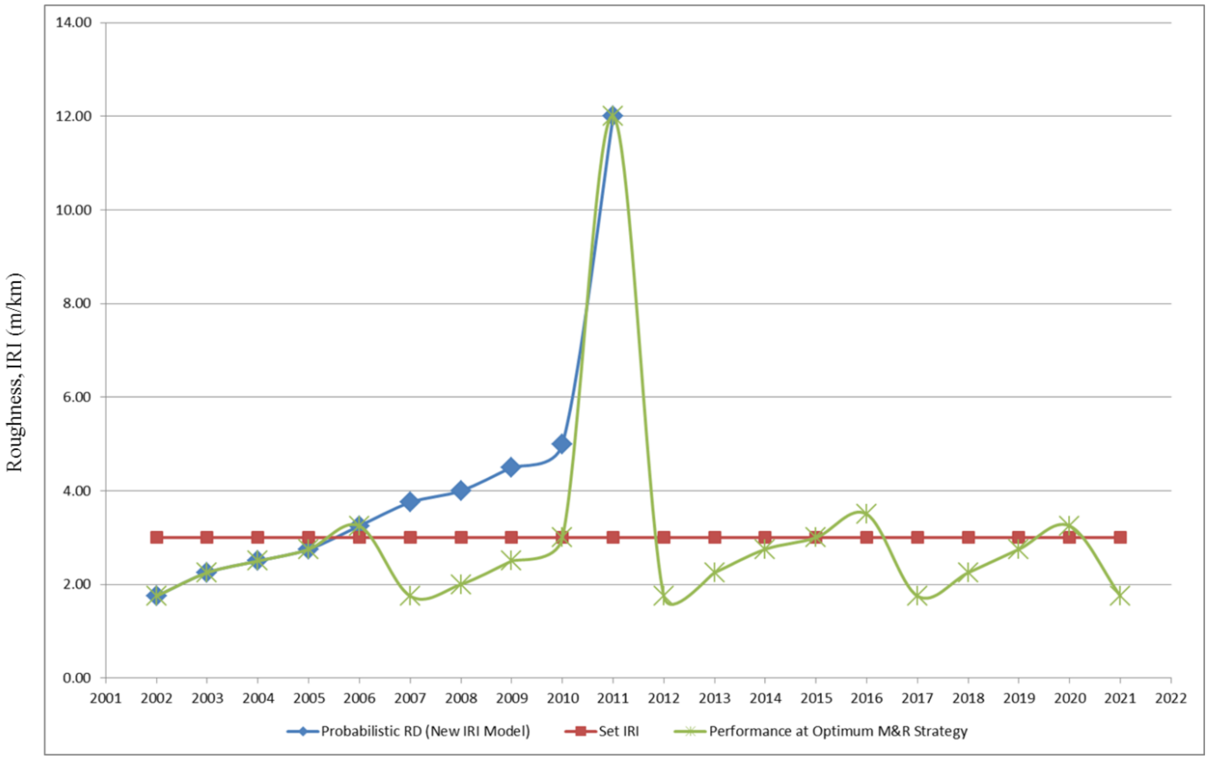


Figure 4.3: Indicative performance of a road group with an optimum M&R strategy using the probabilistic RD models at unconstrained budget

4.4 Network and Project Level RD Models

The RD models were developed for the 27 road groups (network level) and some specific roads in Logan, Queensland (project level). These models assume: i) flooding in year 1, ii) initial road condition as excellent, and iii) no rehabilitation in the next 2 to 3 years. As a result, a deterioration envelop was derived covering 0% to 100% probability of flooding.

As an example, two road groups out of 27 have been chosen here to demonstrate the numerical results of a RD model incorporating flooding. The two road groups are a) Flexible pavement with Low Traffic loading and Strong strength (F_LT_S), and b) Flexible pavement with Low Traffic loading and Poor strength (F_LT_P). Some key features of these road groups are given in *Table 4.1*. Detailed results of all the flexible and composite pavement road groups are shown in *Appendix 4B.*

The TMR-QLD's road database has been used for the twenty seven road groups and derivation of with and without flood TPMs for each road group using the observed roughness and rutting data. The Monte Carlo simulation used the TPMs as inputs and provided the RD models at different probabilities of flooding after simulation. The detailed RD modelling results for all the road groups are shown in *Appendix 4C*.

Figure 4.4 shows the developed roughness and rutting-based RD models for the two road groups. A comparison is given between observed and predicted pavement performances using the roughness and rutting data. The observed each year's with and without flooding data were averaged from the respective yearly flooding and non-flooding data. In *Figure 4.4*, observed non-flooding data is shown in year 1 and a flood is considered in year 2. However, the simulation considered a flood in year 1, hence, the RD models with flooding show abrupt jumps at the first year. Therefore, the prediction and observed results do not match accurately although their trends seem to be reasonable. It was observed that the roughness and rutting prediction due to flooding (at 2nd year) were closer with the observed data for the F_LT_P. It is worth mentioning that 5.0 IRI and 25 mm rutting have been considered as the ultimate values in the analysis as practised by the TMR-QLD.

Figure 4.5 shows the pavement performances with flooding (using IRI and rutting vs. time) at different flooding probabilities for the two road groups. All of these have been shown starting from year 1; nevertheless, a roughness increase was not well predicted at year 1 for F_LT_S. These results reveal that the highest impact on pavement performances up to a certain period may be observed at the highest probability of flooding, i.e., the higher the probability of flooding, the poorer is the pavement performance. For example, a 50% probability of flooding has more effect on pavement performance than a 10% probability of flooding. The Monte Carlo simulation has been used for all these predictions. It has been noted that after a certain period the RD models became flat, therefore, the first few year's results were used to show the RD model's scenario with different flooding probabilities.

Figure 4.5a provides a high roughness impact range at different flooding probabilities for the F_LT_S; whereas, a low impact range is observed for the F_LT_P. This is contrary when rutting-base RD models were assessed, where the F_LT_P road group has a higher impact range. In general, a higher strength road should perform better than a poor road. The roughness-based RD models for the two road groups reveal that if the year 1 impact is considered then the F_LT_S road group performs better compared to the F_LT_P. However, it was not valid for the remaining few years where the stronger pavement has less deterioration up to 10% probability of flooding, and beyond that it performs poorer. It is worth mentioning that the stronger road performs better at different flooding probabilities when rutting-based RD models were compared (see *Figures 4.5b* and 4.*5d*).

Figure 4.6 shows ΔIRI and ΔRutting vs. time at different probabilities of flooding for the F_LT_S road group, which are presented to show pavement performances for year 1 and first 3 years. In general, the highest probability of flooding has the highest effect, and the lowest probability has the lowest impact on pavement performances. Roughness increase in year 1 at different probabilities of flooding lies in between 0.48 and 0.58 IRI, and rutting was increased in the range of 0.66 to 3.37 mm. As an example, 0.48 IRI, 0.49 IRI, 0.53 IRI and 0.58 IRI increase in roughness were observed in year 1 at 0%, 10%, 50% and 100% probability of flooding events, respectively. Similarly, 0.66 mm, 0.94 mm, 2.02 mm and 3.37 mm rutting increase were found respectively at 0%, 10%, 50% and 100% probability of flooding.

Similar to the above one, ΔIRI and ΔRutting vs. time at different probabilities of flooding for the F_LT_P are given in *Figure 4.7*. Pavement performance trends with flooding were the same for both the road groups. Roughness increase in year 1 at different probabilities of flooding was observed in between 0.62 and 0.85 IRI, and rutting was increased in the range of 0.92 to 3.53 mm. As an example, 0.62 IRI, 0.64 IRI, 0.73 IRI and 0.85 IRI increase in roughness were observed in year 1 at 0%, 10%, 50% and 100% probability of flooding events, respectively. Similarly, 0.92 mm, 1.16 mm, 2.22 mm and 3.53 mm rutting increase were found respectively at 0%, 10%, 50% and 100% probability of flooding.

These reveal that the ΔIRI and ΔRutting were increased at higher rates for the poorer pavement; as a result, the stronger road performs better during flooding. The ΔIRI and ΔRutting values help obtaining pavement performances and provide results on flood resilience. Change of IRI and rutting values between a specific probability of flooding and the normal condition give a flood consequence result. Moreover, a road authority can plan for an appropriate treatment as an after-flood rehabilitation using the predicted deterioration after a flood and available resources, and may ensure necessary budget at the right time. All these help in strategic planning for sustainable road asset management. Moreover, investigation of this inherent roughness and rutting increase due to flooding would also be useful.

Similarly, the site specific RD models were also derived for some selected roads in Logan, Queensland. As an example, *Figure 4.8a* shows the new roughness-based RD model for the Flexible pavement with High Traffic loading and Poor strength (F_HT_P) part of Mount Lindesay Highway in Logan, and *Figure 4.8b* for the Flexible pavement with Medium Traffic loading and Fair strength (F_MT_F) part of Beaudesert-Beenleigh Road.

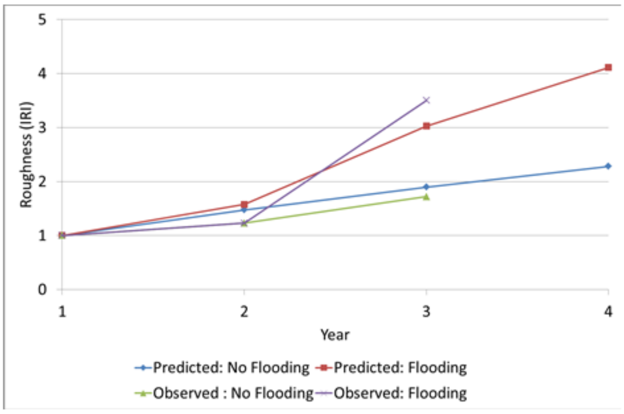
These results are important in selecting an optimum treatment for an after flood rehabilitation for a specific road group. For example, using the roughness-based RD model with 50% flooding probability for the F_LT_S, the second year roughness was found 1.43 IRI. If funding is ensured at the second year, then the optimum treatment (derived from a PMS tool like HDM-4) may be a thin overlay to ensure that the road condition is good. However, if funding is not secured until the third year due to budget constraints, which is a reality, then the roughness-based RD model for this road group with 50% flooding probability shows that the new roughness value would be 2.30 IRI at that year. As a result, the selected thin overlay may not be appropriate in the third year as the road has further deteriorated. Therefore, the needed treatment may be a thick overlay to improve the road condition. Similarly, the rutting-based RD models may be used together with the roughness-based RD models for appropriate treatment selection for a road group. The project level RD models also provide the similar benefits in selecting appropriate treatments.

It has been noted that TMR-QLD did not receive all the necessary funding in the second year (in 2012) for the after flood rehabilitation in Queensland. They managed to complete rehabilitation of the January 2011 flood damaged roads in 3 to 4 years' time. Therefore, in practice, it is essential to use the RD models with different flooding probabilities for a specific road group to determine current road condition states at the second, third and fourth years, etc. so that an appropriate treatment is chosen for after flood rehabilitation.

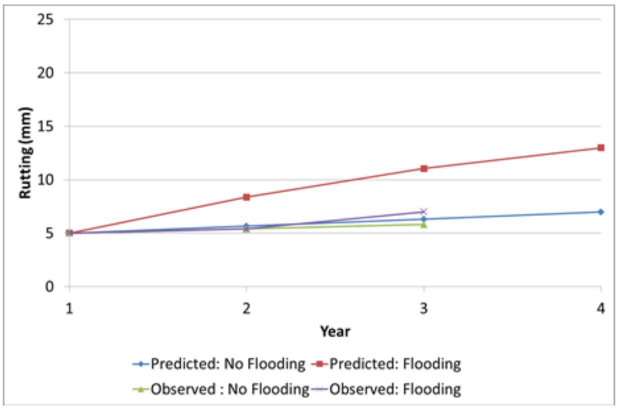
Table 4.1: Key features of the two road groups chosen in the analysis

Road Groups	**Description**	**Length (km) and Width (m)**	**Seal Age (year) and Pavement Age (year)**	**Pavement Depth (mm) and Wearing Course layer (mm)**	**AADT* and % HV***	**ESAL***	**Strength No. = (1/seal age) * (1/pavement age) * (pavement depth)**	**Base Year IRI (m/km)**
F_LT_S	Flexible, Low Traffic loading and Strong strength	1,534 km and 7.5 m (1+1 m shoulder)	2.24 and 14.16 years	223 and 50 mm	230 and 22.59%	374,996	48.95	2.81
F_LT_P	Flexible, Low Traffic loading and Poor strength	4,399 km and 7.5 m (1+1 m shoulder)	7.94 and 35.68 years	139 and 50 mm	250 and 21.24%	403,170	0.57	3.55

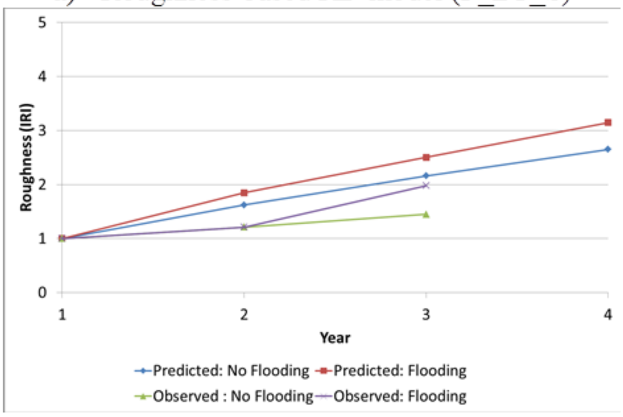
*AADT = Annual Average Daily Traffic, HV = Heavy Vehicle, and ESAL = Equivalent Single Axle Loading



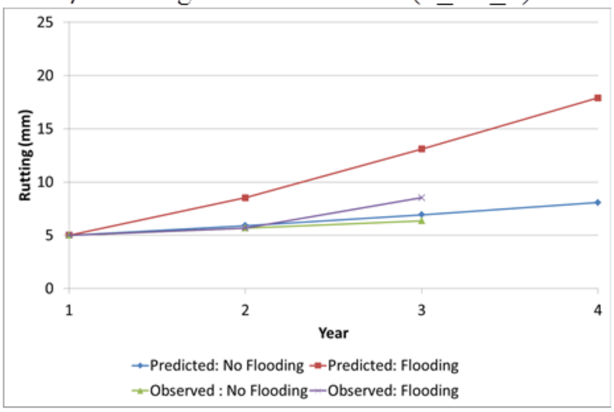
a) Roughness-based RD model (F_LT_S)



b) Rutting-based RD model (F_LT_S)

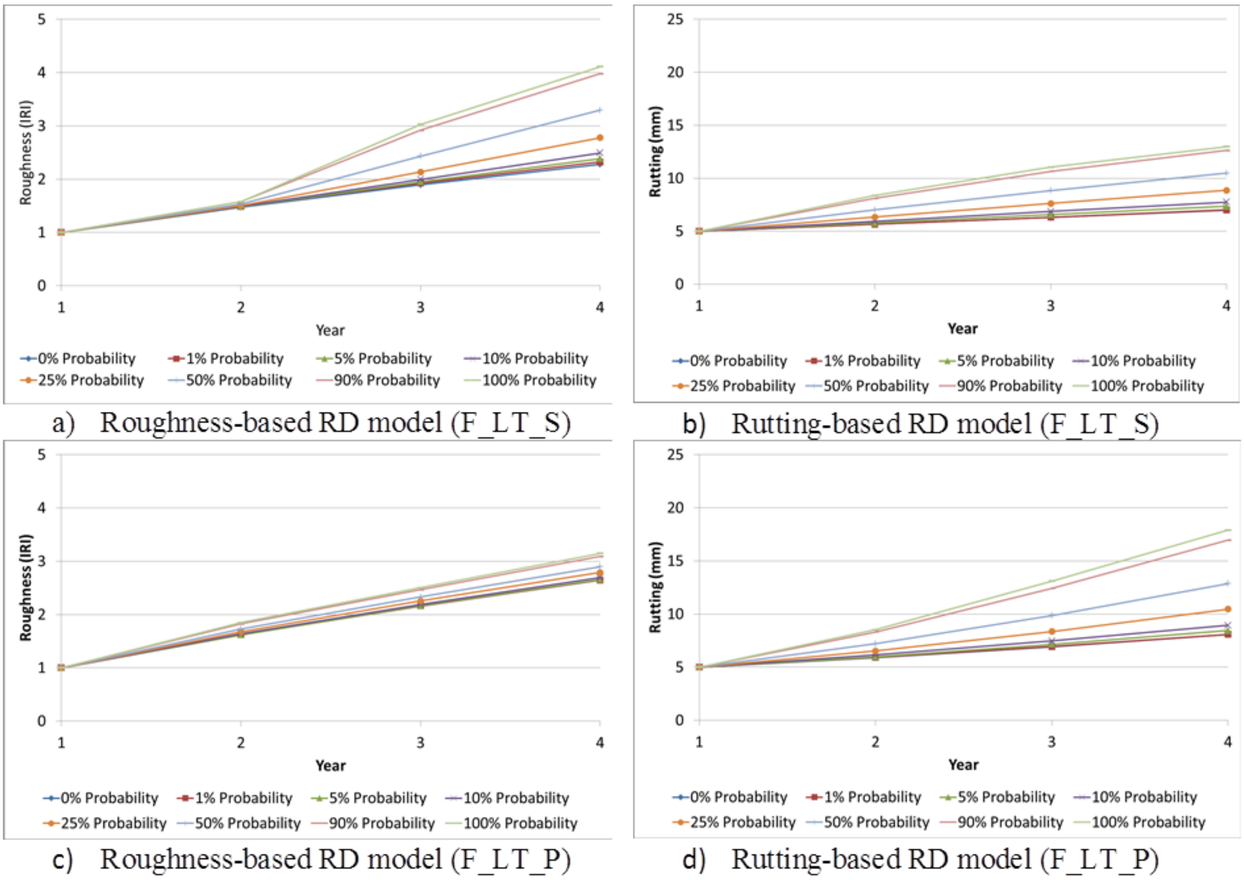


c) Roughness-based RD model (F_LT_P)



d) Rutting-based RD model (F_LT_P)

Figure 4.4: Roughness and rutting-based RD models with observed data for two road groups



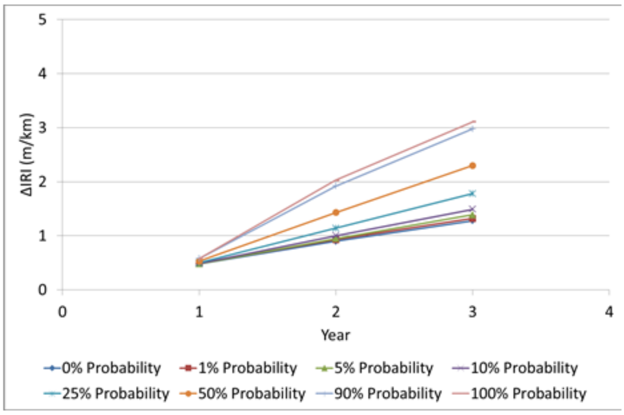
a) Roughness-based RD model (F_LT_S)

b) Rutting-based RD model (F_LT_S)

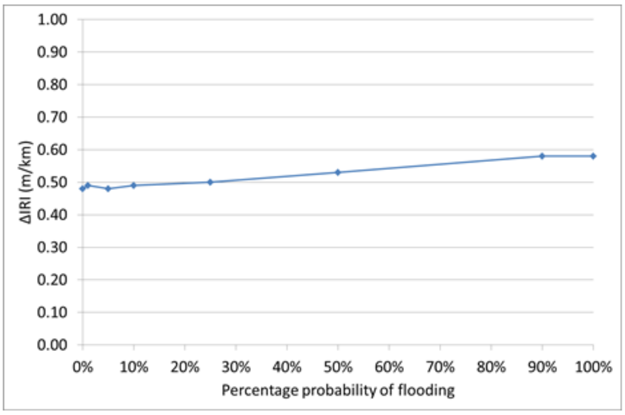
c) Roughness-based RD model (F_LT_P)

d) Rutting-based RD model (F_LT_P)

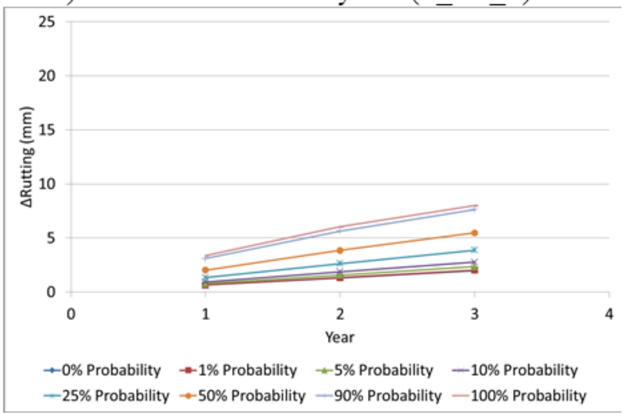
Figure 4.5: Roughness and rutting-based RD models with different flooding probabilities for two road groups



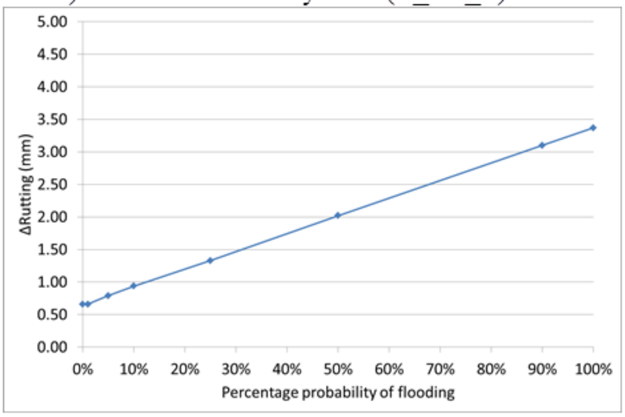
a) ΔIRI vs. time for 3 years (F_LT_S)



b) ΔIRI vs. time at year 1 (F_LT_S)

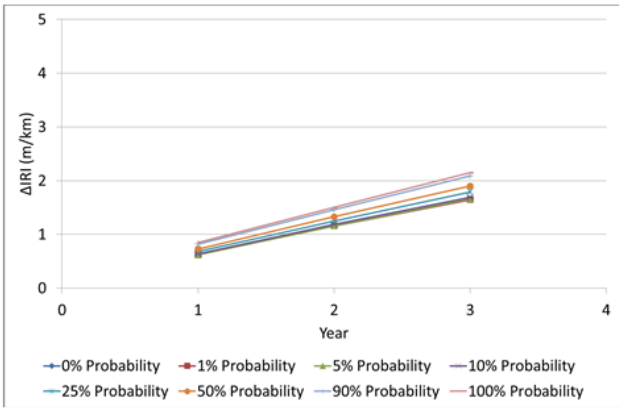


c) ΔRutting vs. time for 3 years (F_LT_S)

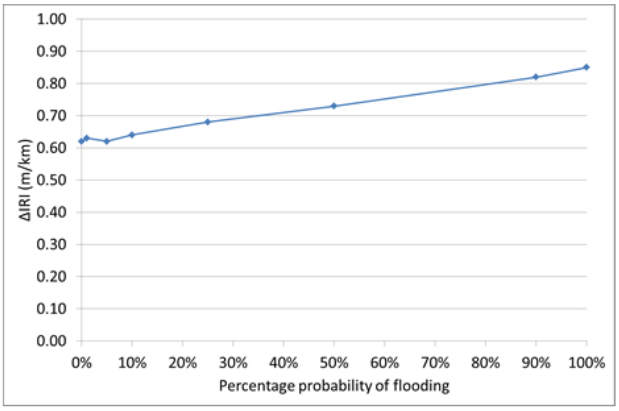


d) ΔRutting vs. time at year 1 (F_LT_S)

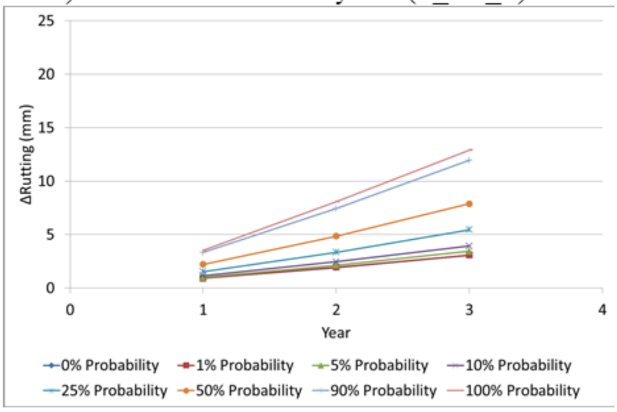
Figure 4.6: ΔIRI and ΔRutting vs. time due to different probabilities of flooding for the F_LT_S road group



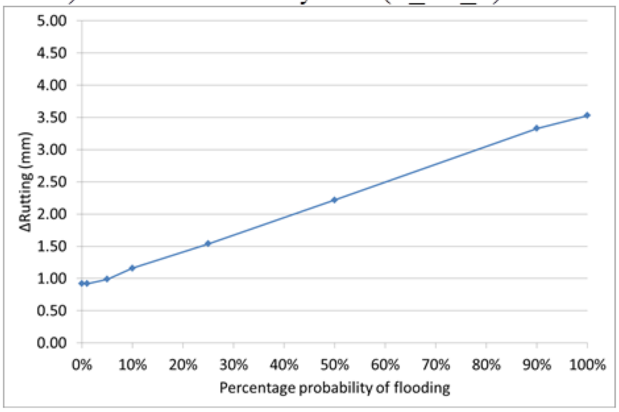
a) ΔIRI vs. time for 3 years (F_LT_P)



b) ΔIRI vs. time at year 1 (F_LT_P)

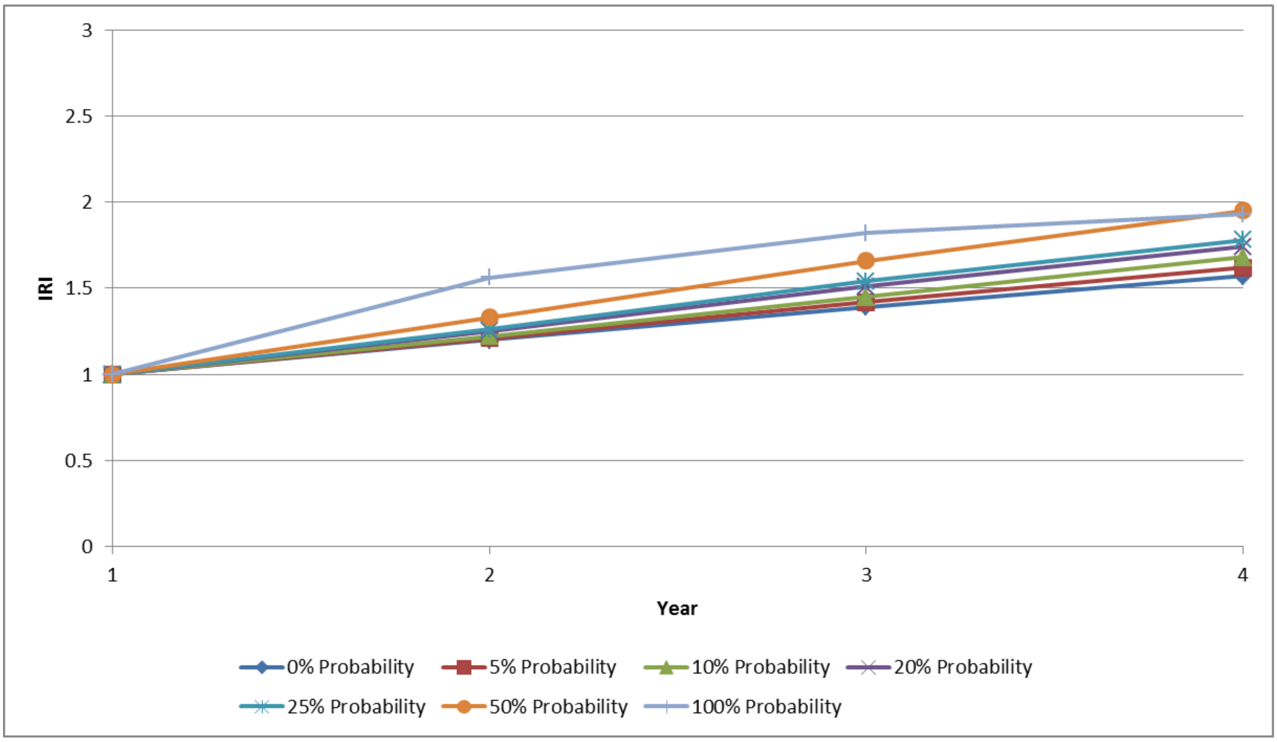


c) ΔRutting vs. time for 3 years (F_LT_P)



d) ΔRutting vs. time at year 1 (F_LT_P)

Figure 4.7: ΔIRI and ΔRutting vs. time due to different probabilities of flooding for the F_LT_P road group



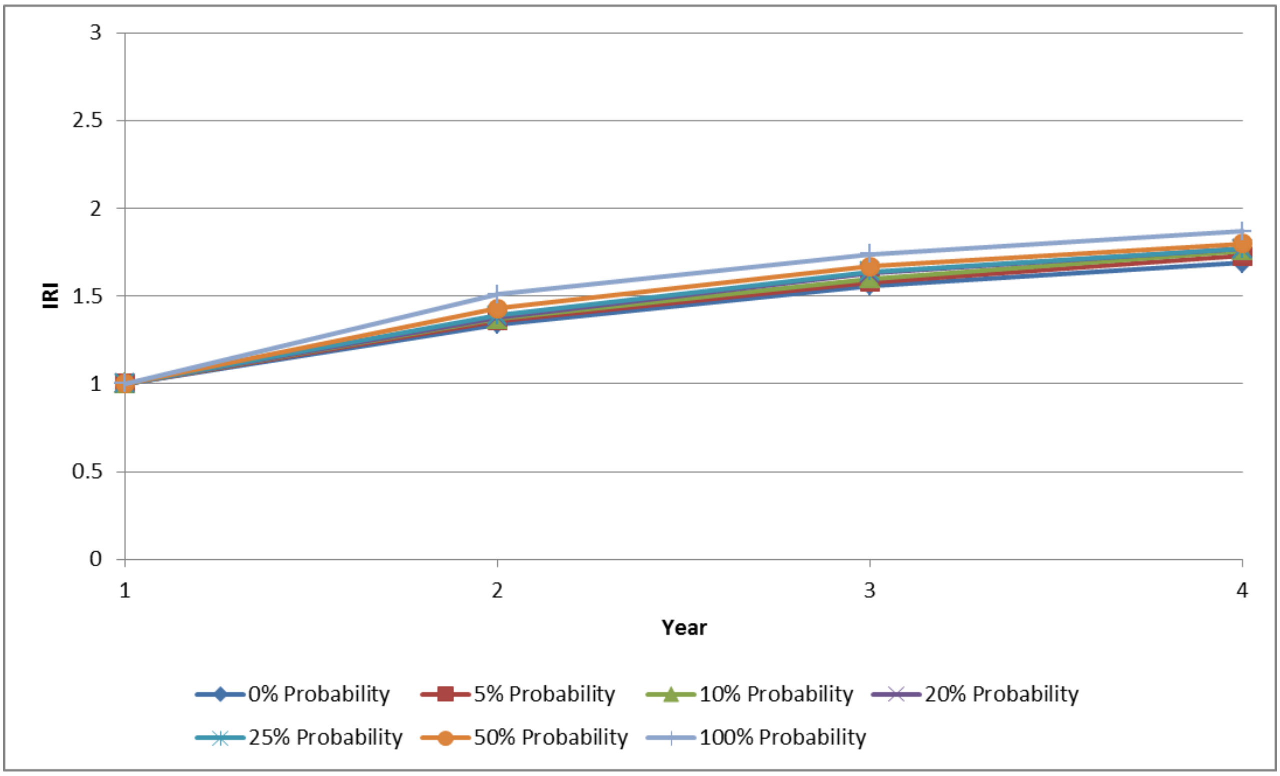


Figure 4.8: Site specific roughness-based RD models for a) F_HT_P part of Mount Lindesay Highway and b) F_MT_F of Beaudesert-Beenleigh Road

4.5 Validation of the RD Models

The derived RD models were validated with the actual after flood data for some selected roads in Logan which is located at the South-Eastern part of Queensland. There are about 23 major roads under this city being managed by the TMR-QLD (LCC, 2015a). In the current analysis, all of these roads were checked whether they were affected in the January 2011 flooding. After investigation using the 20%, 50% and 100% probability flooding maps, it appears that the following four major roads were partially damaged in the assessed flooding:

- Mount Lindesay highway,
- Beaudesert-Beenleigh road,
- Waterford-Tamborine road, and
- Pacific Highway (service road).

The TMR-QLD's road network in Logan city is shown in *Appendix 4D*. The 20%, 50% and 100% probability of flooding maps for Logan (LCC, 2015b) were used to assess impact of the January 2011 flooding on these four roads. The flooding maps for Logan are given in *Appendix 4E*. As an example, a partly damaged of the Mount Lindesay Highway at 50% flooding probability is shown in *Figure 4.9*. Finally, the short-term RD models were generated for these roads.

Major flood records for the Albert and Logan rivers have been checked for Logan (BoM, 2014), which is shown in *Appendix 4F*. It seems that the January 2011 flooding level was 7.05m at Waterford area in Logan, which is termed as minor at the higher side. *Appendix 4F* reveals that a Major flood could be in between 1 and 5% probability of flooding, a moderate flood ranges within 5 to 10% probability, and a minor flood ranges from 10 to 30% probability. Therefore, considering the flood level at Waterford, the January 2011 flooding may be considered as a 20% probability of flooding.

The above-mentioned four roads are within the representative road groups developed for the TMR-QLD's road network. The estimated affected length derived from the 20% probability of flooding map is given in *Table 4.2*.

The newly derived representative roughness and rutting based RD models were used to predict the after-flood road deterioration for these roads. The following approach was used to obtain ΔIRI and

ΔRutting at 20% probability of flooding (see *Figure 4.10* for roughness), which is given in *Equations 4.1 and 4.2*. The *Equation 4.1* is depicted in *Figure 4.10*.

$$\Delta IRI_{f20\%} = \Delta IRI_{20\%} - \Delta IRI_{0\%} \tag{4.1}$$

Where,
$\Delta IRI_{20\%}$ = incremental change in roughness at time t due to a 20% probability of flooding,
$\Delta IRI_{0\%}$ = incremental change in roughness at time t due to a 0% probability of flooding (i.e. at normal condition), and
$\Delta IRI_{f20\%}$ = incremental change in roughness at time t due to a 20% probability of flooding only.

$$\Delta Rutting_{f20\%} = \Delta Rutting_{20\%} - \Delta Rutting_{0\%} \tag{4.2}$$

Where,
$\Delta Rutting_{20\%}$ = incremental change in rutting at time t due to a 20% probability of flooding,
$\Delta Rutting_{0\%}$ = incremental change in rutting at time t due to a 0% probability of flooding (i.e. at normal condition), and
$\Delta Rutting_{f20\%}$ = incremental change in rutting at time t due to a 20% probability of flooding only.

Table 4.2 reveals that the affected part of the Mount Lindesay Highway falls under a representative road group of a Flexible pavement with Medium Traffic loading and a Strong strength level (F_MT_S). Thus, the derived roughness and rutting-based RD models for this road group were used for the Mount Lindesay Highway. Using the approach shown in *Figure 4.10* to determine ΔIRI and ΔRutting at a specific time because of only 20% probability of flooding, $\Delta IRI_{f20\%}$ vs. time and $\Delta Rutting_{f20\%}$ vs. time were generated for short period for the effected part of the Mount Lindesay Highway. The estimated ΔIRI vs. time (month) at 0% and 20% probability of flooding along with $\Delta IRI_{f20\%}$ vs. time (month) were shown in *Figure 4.10*. It reveals that a 20% probability of flooding will provide more damage with time. In fact, the higher the probability of flooding, the poorer is the pavement's performance. As a result, $\Delta IRI_{f20\%}$ increases every month.

The predicted deterioration due to the January 2011 flooding for the Mount Lindesay Highway has been compared with the actual data. Actual roughness data was used before the January 2011 flooding, which has been obtained from the TMR-QLD's road database along with engineering judgment to get the value. Moreover, the actual data of April 2011 and February 2012 for the flood affected part were also used. *Figure 4.11* clearly shows that the actual deterioration for the flood affected part of the Mount Lindesay Highway was close to the deterioration predicted at the 20% probability of flooding. Therefore, it is concluded that the new RD models with flooding can closely provide real field deterioration. It also confirms the new RD models.

Month-wise deterioration was predicted at 0% and 20% probability of flooding using the derived representative RD model. The before flooding roughness was 3.05 IRI, which increased to 3.25 IRI in a year after the flooding. If the RD model is not used for a 20% probability of flooding, then a lower roughness of 3.19 IRI would be obtained. This may influence in wrong treatment selection; for example, a thinner overlay may be chosen instead of a thicker one.

Results of the Mount Lindesay Highway show that road deterioration considering roughness at 20% probability of flooding was slightly higher than the normal condition. As this road is a strong strength one and well maintained, the deterioration rate was found low. Weaker roads would have higher deterioration rate after a flood. Therefore, an appropriate road deterioration prediction after a flood is essential for right treatment selection.

Similar relationships were generated with Rutting vs. time (month) and $\Delta\text{Rutting}_{f20\%}$ vs. time (month) after the January 2011 flooding, which is given in *Figure 4.12*. It reveals that before the flooding rutting of that part was 5.21 mm, and after the flooding in a year time rutting would be 6.37 mm. Without an RD model with flooding, a road authority would predict 5.70 mm of rutting after one year. It indicates incorrect deterioration prediction, and ultimately wrong treatment selection as post flood rehabilitation.

Figure 4.12 reveals that the April 2011 rutting value was close to the predicted rutting at 20% probability of flooding. However, a reduced rutting was found in February 2012 which indicates that any minor maintenance might have been done that reduced rutting. Otherwise, the rutting value of February 2012 cannot be lower than the April 2011 one. The TMR-QLD's database does not have a record of the treatment given before February 2012. Therefore, it was assumed as a minor treatment, as roughness value was not reduced in February 2012 which is shown in *Figure 4.11*.

The derived RD model at 20% probability of flooding, i.e., January 2011 flooding, shows close match with the actual data, which has been shown for the effected part of the Mount Lindesay Highway. Similarly, a close match was observed for the effected part of the Beaudesert-Beenleigh road (see *Appendix 4G*). As a result, it appears that the new RD models are consistent to the actual deterioration for short period up to 2 years. *Appendix 4G* provides results for the Beaudesert-Beenleigh road, which possess the same road group of Pacific Highway (service road). Therefore, the Pacific Highway (service road) would have the same deterioration trend. However, it was not possible to assess this road because of not having real after flood data. The Waterford-Tamborine Road belongs to the F_MT_F road group, and their RD models are not acceptable due to incorrect data. Therefore, no RD models were generated for this road.

The project level roughness-based RD models were also validated with the actual after January 2011 flood data, and found acceptable. The models were also compared with the representative road group' RD models. As an example, *Figure 4.13* shows the site specific RD model at 20% probability of flooding which was same to the January 2011 flood. The actual after flood data and the same road group's (F_MT_S) RD model were also compared. The results reveal that both the site specific and network level RD models were consistent to the actual field condition for up to 2 years. Here, year 1 means January 2011 just before the flood. More detailed results are given in *Appendix 4H*.

The new network and site-specific RD model results were also checked using a t-test, which requires a null hypothesis and an alternative hypothesis. If the null hypothesis is accepted, then the alternative one is rejected; and vice versa. The analysis assumed a null hypothesis that there is no difference between means of the two data sets. The p-value supports the null hypothesis; if a p-value is high than data are likely with a true null. In all the cases, statistically insignificant differences were observed between the predicted deterioration using the RD models and actual data (see *Table 4.3*). Therefore, these RD models are appropriate for future use.

Table 4.2: Estimated effected length due to the January 2011 flooding

Road Name	Length in Logan Area (km)	Representative Road Groups	Road Length Effected by the January 2011 Flooding or 20% Probability of Flooding (km)
Mount Lindesay Highway	**58.06**	**F_MT_S**	**0.45**
Beaudesert-Beenleigh Road	**33.46**	**C_HT_S**	**0.40**
Waterford-Tamborine Road	**24.97**	**F_MT_F**	**1.50**
Pacific Highway (Service Road)	**1.55**	**C_HT_S**	**0.60**

a. F_MT_F means Flexible pavement, Medium Traffic loading and Fair strength road group
b. C_HT_S means Composite pavement, High Traffic loading and Strong strength road group

Table 4.3: Validation of the network and site-specific RD models' results with the actual data and t-tests for some major roads in Logan

Road Name	Road Group	Actual Roughness Data for the Flood Effected Part	Network Roughness based RD Model results	t-test Results (Network RD Model Result vs. Actual Data)	Actual Rutting Data for the Flood Effected Part	Network Rutting based RD Model results	t-test Results (Network Rutting based RD Model Result vs. Actual Data)	Site-specific Roughness based RD Model results	t-test Results (Site-specific RD Model Result vs. Actual Data)
Mount Lindesay Highway	F_MT_S	Jan 11: 3.05 IRI Feb 12: 3.31 IRI	Jan 11: 3.05 IRI Feb 12: 3.28 IRI	Two tailed P value: 0.9390 t: 0.0864 Degrees of freedom (Df): 2 Not statistically significant	Jan 11: 5.21 mm April 11: 5.46 mm	Jan 11: 5.21 mm April 11: 5.50 mm	Two tailed P value: 0.9263 t: 0.1045 Df: 2 Not statistically significant	-	-
Beaudesert-Beenleigh Road	C_HT_S	Jan 11: 1.62 IRI Dec 12: 2.57 IRI	Jan 11: 1.62 IRI Dec 12: 2.59 IRI	Two tailed P value: 0.9896 t: 0.0147 Df: 2 Not statistically significant	Jan 11: 3.71 mm Dec 12: 5.57 mm	Jan 11: 3.71 mm Dec 12: 5.54 mm	Two tailed P value: 0.9919 t: 0.0115 Df: 2 Not statistically significant	-	-
Beaudesert-Beenleigh Road	F_MT_P	Jan 11: 1.07 IRI Feb 12: 1.30 IRI	-	-	-	-	-	Jan 11: 1.07 IRI Feb 12: 1.41 IRI	Two tailed P value: 0.8138 t: 0.2680 Df: 2 Not statistically significant
Beaudesert-Beenleigh Road	F_MT_F	Jan 11: 1.25 IRI Feb 12: 1.46 IRI	-	-	-	-	-	Jan 11: 1.25 IRI Feb 12: 1.63 IRI	Two tailed P value: 0.7332 t: 0.3916 Df: 2 Not statistically significant
Waterford-Tamborine Road	F_MT_F	Jan 11: 2.00 IRI Feb 12: 2.19 IRI Dec 12: 2.32 IRI	-	-	-	-	-	Jan 11: 2.00 IRI Feb 12: 2.09 IRI Dec 12: 2.18 IRI	Two tailed P value: 0.4942 t: 0.7515 Df: 4 Not statistically significant

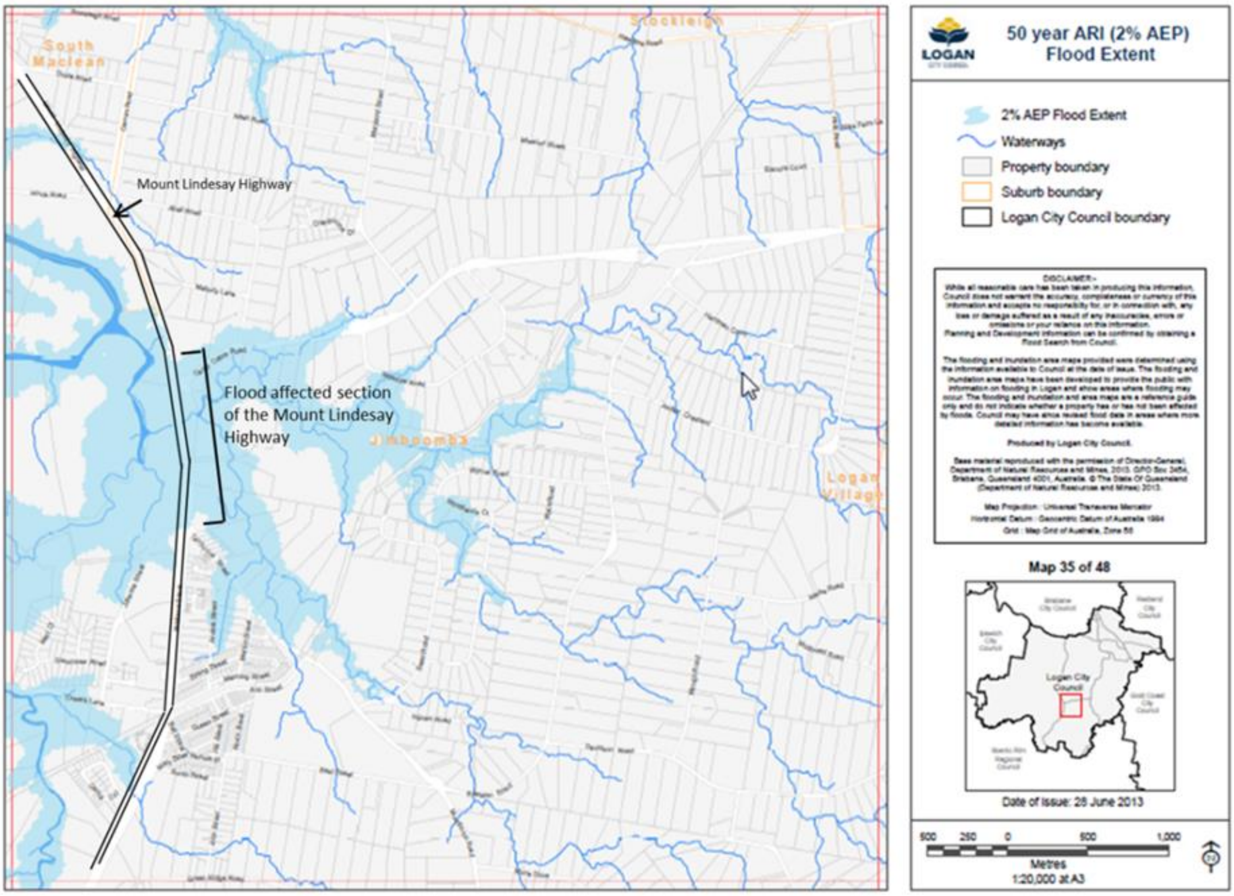


Figure 4.9: Flood affected Mount Lindesay Highway at 50% flooding probability (LCC, 2015b)

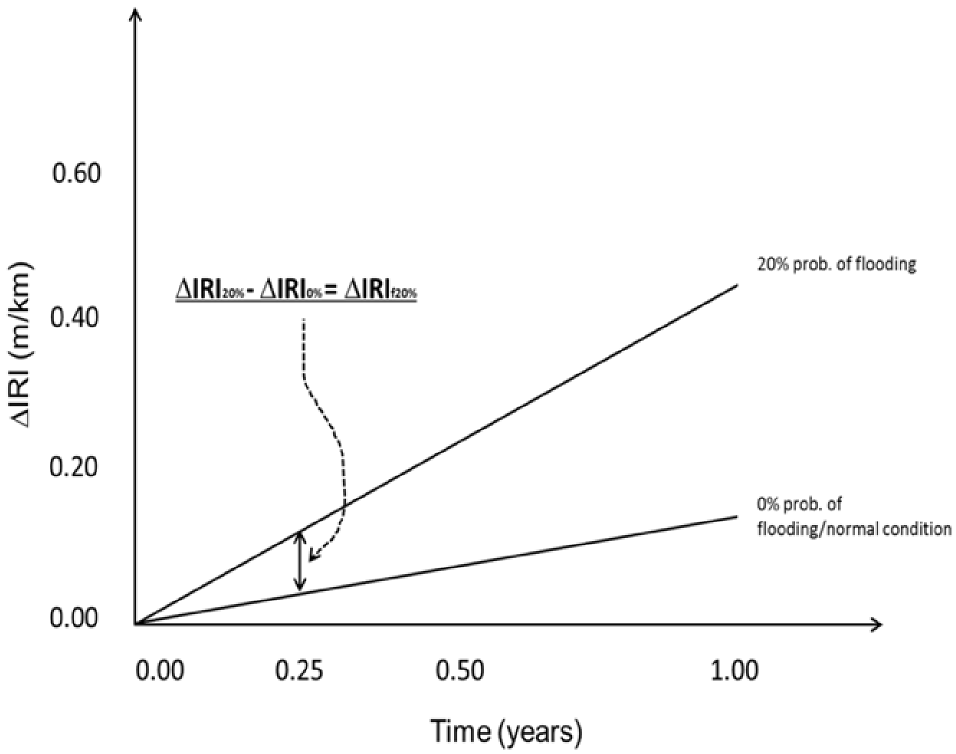


Figure 4.10: ΔIRI vs. time at different probabilities of flooding

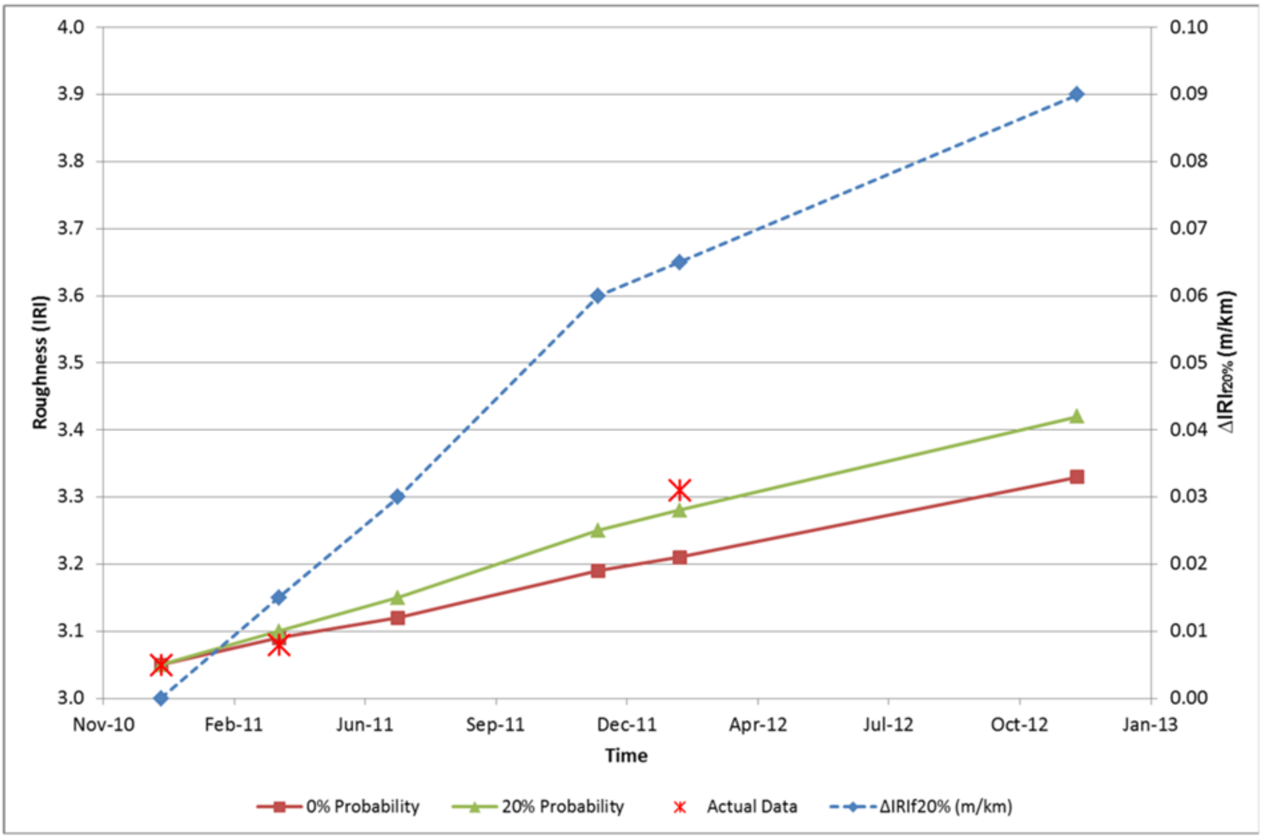


Figure 4.11: IRI vs. time and ΔIRIf20% vs. time due to the January 2011 flooding for a part of the Mount Lindesay Highway

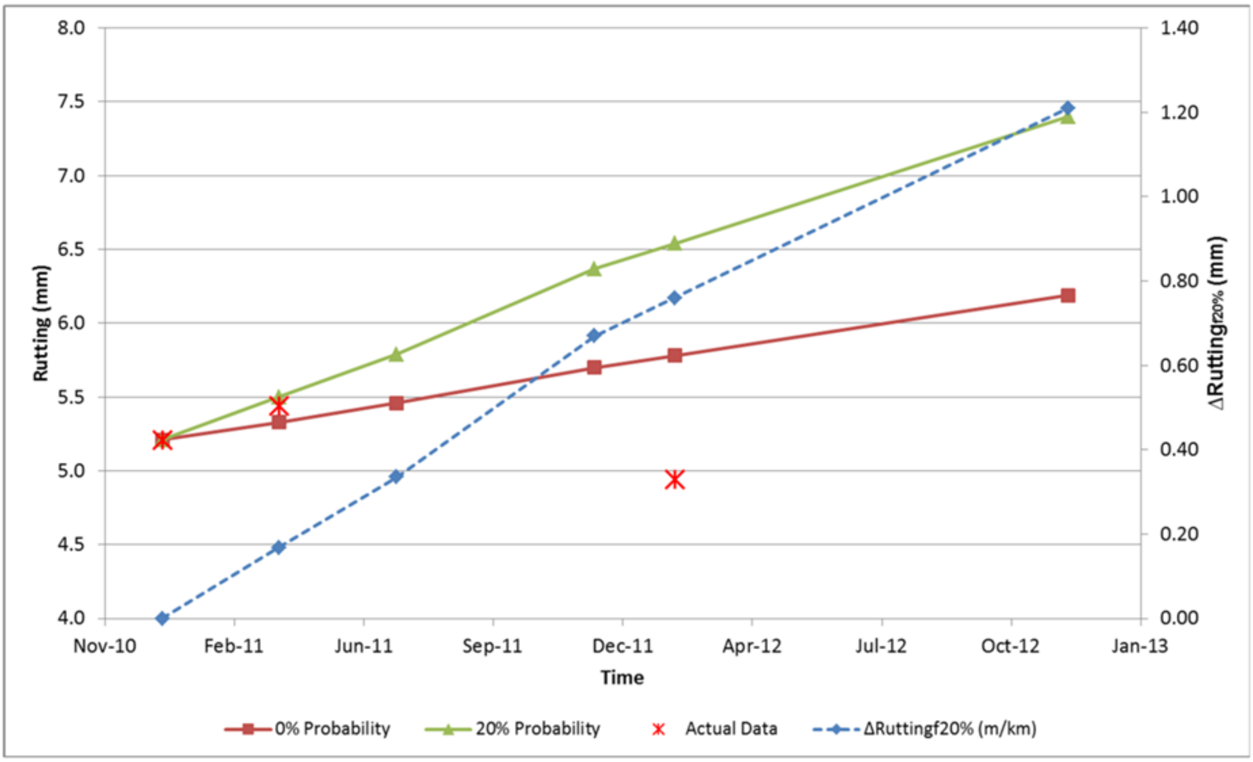


Figure 4.12: Rutting vs. time and ΔRuttingf20% vs. time due to the January 2011 flooding for a part of the Mount Lindesay Highway

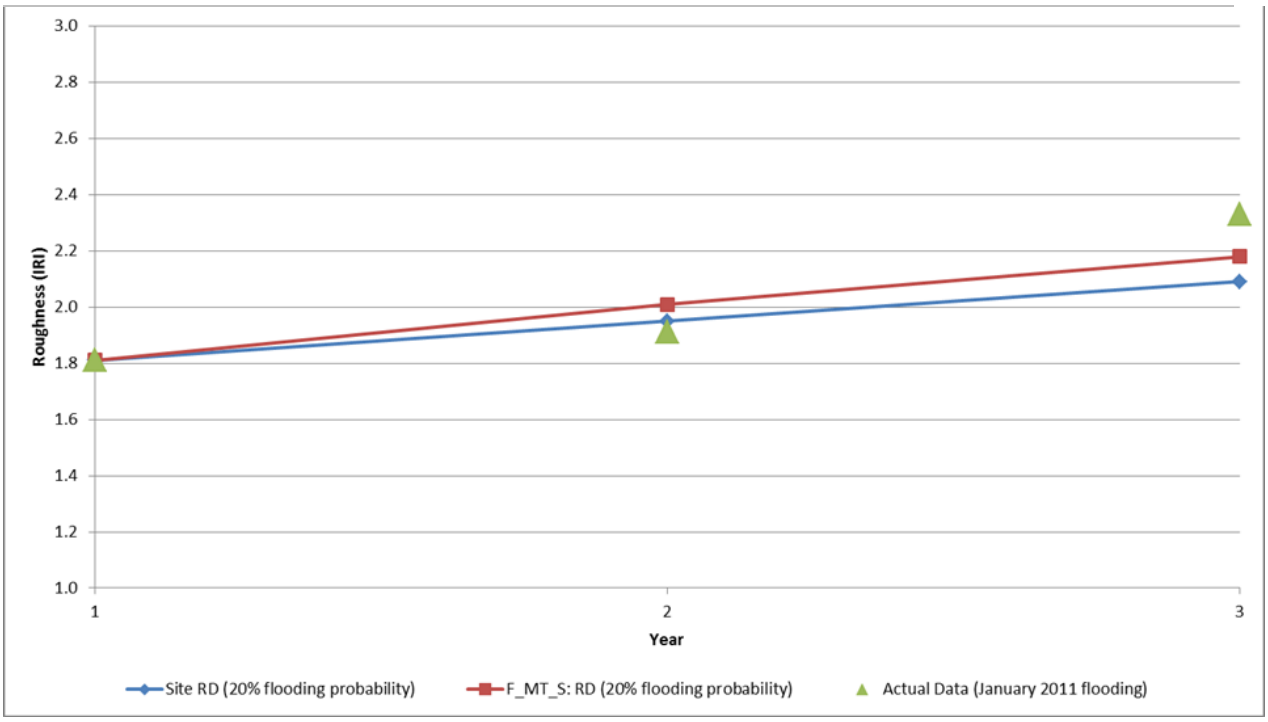


Figure 4.13: Comparison of the project and network levels RD models with actual data for the F_MT_S part of the Waterford-Tamborine Road

4.6 Summary

This chapter discusses development and results of the new roughness and rutting based RD models with flooding. It shows the key inputs, indicative and detailed RD modelling results. The RD models were derived at different probabilities of flooding at the network and project levels. It shows that the higher the probability of flooding, the poorer is the pavement performance up to a certain period which is 2 to 3 years. These site-specific and network levels RD models were validated with the actual data obtained for some major roads in Logan and with t-tests. The predicted road deterioration after a flood closely matched with the observed data. The t-tests results showed that the differences in predictive and observed results are not statistically significant. Therefore, the new RD models with flooding are suitable in selecting appropriate treatments for road asset preservation after a flood. Their importance in treatment selection is also highlighted.

CHAPTER FIVE: IMPACT OF Mr LOSS ON PAVEMENT PERFORMANCES

5.1 Introduction

Moisture intrusion in pavements because of a flood reduces Mr loss at granular and subgrade layers. As a result, pavements perform poorly. Two popular roughness models were used to obtain the impact of Mr loss on pavement performances. These results have verified consistency of the RD models. Details of the theoretical part were discussed in Section 3.2, and the approach in assessing pavement performances due to Mr loss at untreated layers has been shown in *Figure 3.5*. The components shown in this chapter is given in *Figure 3.1*.

5.2 Application of the Framework

Seven representative flexible pavement road groups were chosen in the analysis, which covered about 49.6% of the Queensland's 34,000 km major road network. They were selected to assess impact of a flood on different pavement performances. These road groups differ in length, pavement strength (relates to layer thicknesses, seal age and pavement age), AADT, traffic loading and initial IRI. Road groupings were shown in Section 3.1, and detailed of these seven road groups are given in *Table 5.1*. It is important to note that base and sub-base layers thicknesses were assumed in the analysis, which had an impact on the SN values.

The ESALs are shown for year 1 and 20 years design period (see *Table 5.1*). In the current research, traffic loading was assumed the same before and after a flood. The impact of moisture change on granular and subgrade layers' modulus can be captured for some weeks using the EICM model. However, the AASHTO 2008 and HDM-4 roughness models predict deterioration in yearly basis. While deriving probabilistic road deterioration models with flooding using the above dataset reveals that a pavement performance is affected for some initial years because of a flood. Therefore, ΔIRI in year 1 has been used to represent impact of a flood on pavement performances.

Table 5.1: Key features of the road groups chosen in the analysis

Road Groups	Description	Length (km) & Width (m)	Seal Age (year) & Pavement Age (year)	Pavement Depth (mm) & Wearing Course layer (mm)	AADT & %HV	Design ESAL	ESAL in Year 1	Strength No. (1/seal age * 1/pavement age * pavement depth)	Base Year IRI (m/km)
F_LT_P	Flexible, Low Traffic loading & Poor strength	4,399 km & 7.5 m (1+1 m shoulder)	7.94 & 35.68 years	139 & 50 mm	250 & 21.24%	403,170	16,959	0.57	3.54
F_LT_F	Flexible, Low Traffic loading & Fair strength	3,696 km & 7.5 m (1+1 m shoulder)	4.73 & 24.89 years	192 & 50 mm	262 & 21.52%	399,671	18,007	2.29	3.09
F_MT_F	Flexible, Medium Traffic loading & Fair strength	4,313 km & 9 m (1+1 m shoulder)	5.6 & 24.06 years	231 & 50 mm	2,314 & 19.67%	3,540,634	145,368	2.27	2.87
F_MT_S	Flexible, Medium Traffic loading & Strong strength	2,533 km & 9 m (1+1 m shoulder)	2.78 & 13.11 years	303 & 50 mm	2,054 & 20.63%	3,577,461	135,332	98.47	2.58
F_HT_P	Flexible, High Traffic loading & Poor strength	541 km & 12 m (1.5 +1.5 m shoulder)	10.34 & 33.38 years	179 & 100 mm	9,878 & 15.45%	14,500,000	487,414	0.57	2.85
F_HT_F	Flexible, High Traffic loading & Fair strength	967 km & 12 m (1.5+1.5 m shoulder)	6.95 & 22.08 years	284 & 100 mm	9,864 & 16.43%	15,400,000	517,597	2.33	2.47
F_HT_S	Flexible, High Traffic loading & Strong strength	425 km & 12 m (1.5+1.5 m shoulder)	3.77 & 10.75 years	400 & 100 mm	8,996 & 16.60%	14,202,184	476,934	52.13	2.20

Key factors used in the AASHTO 2008 and HDM-4 roughness models were shown in *Table 3.1*. The material properties chosen in the analysis are valid for Australia, especially Queensland. They were assumed from the current Austroads guidelines on pavement structural design (Austroads, 2010; TMR, 2013), which are as follows:

- Subgrade California Bearing Ratio = 5%,
- Subgrade modulus in vertical direction = 50 MPa,
- Sub-base modulus in vertical direction = 125 MPa,
- Base modulus in vertical direction = 500 MPa,
- Tire pressure = 0.75 MPa, and

- Centre to centre tire distance for a dual tire = 92.1 mm.

Results of the analysis are given in *Tables 5.2 to 5.4* and *Figures 5.1 to 5.4*. *Figure 5.1* shows a decrease in SN at different subgrade Mr losses for all the road groups, and the rate of SN reduction becomes higher beyond 30% loss in subgrade Mr. In general, it reveals an obvious relation, i.e., the higher the subgrade Mr loss due to moisture intrusion, the higher is the loss in SN for any road group.

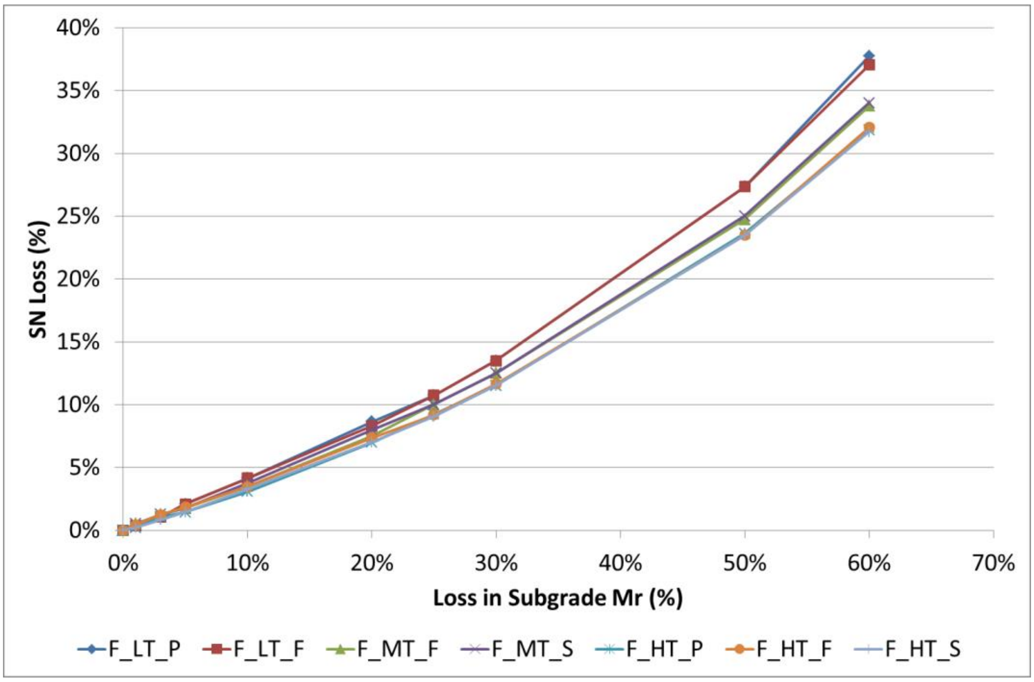


Figure 5.1: Decrease in SN due to loss in subgrade Mr

As mentioned in *Figure 3.5*, Mr losses at granular and subgrade layers due to moisture intrusion were measured using the EICM model (see *Equations 2.9 and 2.10)*. *Table 5.2* shows the Mr loss values of granular and subgrade materials at different percentage change in (S-S_{opt}) using the EICM model. These results were compared with some existing models to validate consistency and found acceptable; which is shown in *Table 5.3*. The previous studies revealed that percentage change in (S-S_{opt}) at without flood condition may rise up to +25% both in granular and subgrade layers (Rada and Witczak, 1981; NCHRP, 2000; Austroads, 2010; Drumm et al., 1997; Jones and Witczak, 1977). The current analysis also shows maximum of +30% increases in (S-S_{opt}) when there is no flood.

After obtaining the Mr loss values using the EICM model, the HDM-4 and AASHTO 2008 roughness prediction models were utilised to obtain ΔIRI at varying Mr losses. The different

Mr values could give respective SN values, which have been used as inputs for the HDM-4 model. On the other hand, the MET has been used to get deflection, ε_c and ε_t values at varying Mr for all of these road groups; which were used as inputs to the AASHTO 2008 roughness prediction models. As expected, pavement deflection, ε_c, and ε_t values increase if Mr decreases, since a pavement becomes weaker when it is saturated. Details can be seen in *Table 5.4*.

Table 5.2: Granular and subgrade layer Mr losses due to moisture intrusion obtained using the EICM model and considered in the current analysis

Change in moisture content, % ($S-S_{opt}$ of *Equation 2.10*)	**Mr loss at granular layers (%)**	**Mr loss at subgrade (%)**
+1%	-2%	-3%
+5%	-11%	-16%
+10%	-21%	-29%
+20%	-34%	-49%
+30%	-42%	-61%

Table 5.3: Comparison of the current Mr loss results with previous studies

($S-S_{opt}$) in %	**Mr/M_{ropt} for Granular Materials**				**Mr/Mr_{opt} for Subgrade**				**Remarks**
	Rada and Witczak, 1981	**NCHRP, 2000**	**Austroads, 2010**	**Current Results with EICM Model**	**Drumm, et al. 1997**	**NCHRP, 2000**	**Jones and Witczak, 1977**	**Current Results with EICM Model**	
+1%				0.98				0.97	The current result matched closely with the existing models.
+5%	0.95	0.90	0.89	0.89	0.85	0.80	0.85	0.84	
+10%	0.90	0.80	0.77	0.79	0.75	0.73	0.70	0.71	
+20%	0.82	0.65	0.59	0.66	0.53	0.50	0.50	0.51	
+30%	-	0.60	-	0.58	-	-	-	0.39	

Table 5.4: Deflection, ε_c and ε_t values at different Mr losses for different road groups derived with the MET method

Granular Mr Loss (%)	Subgrade Mr Loss (%)	Road Groups							
		Deflection (mm) ε_t ε_c	**F_LT_P**	**F_LT_F**	**F_MT_F**	**F_MT_S**	**F_HT_P**	**F_HT_F**	**F_HT_S**
0%	0%	Deflection	0.95	0.87	0.82	0.77	0.66	0.58	0.54
		ε_t	-214.3	-214.3	-214.3	-214.3	-261.5	-261.5	-261.5
		ε_c	2852.4	1884.0	1323.2	849.9	1524.8	832.0	519.3
2%	3%	Deflection	0.97	0.89	0.84	0.79	0.67	0.60	0.55
		ε_t	-218.7	-218.7	-218.7	-218.7	-266.8	-266.8	-266.8
		ε_c	2924.6	1930.8	1355.8	870.6	1562.4	852.3	531.8
11%	16%	Deflection	1.09	1.00	0.93	0.87	0.75	0.66	0.61
		ε_t	-240.8	-240.8	-240.8	-240.8	-293.8	-293.8	-293.8
		ε_c	3292.6	2169.1	1521.3	975.9	1753.9	954.6	595.8
21%	29%	Deflection	1.25	1.14	1.06	0.99	0.86	0.75	0.69
		ε_t	-271.2	-271.2	-271.2	-271.2	-331.0	-331.0	-331.0
		ε_c	3794.4	2494.3	1747.3	1119.8	2015.3	1096.1	683.2
34%	49%	Deflection	1.62	1.45	1.34	1.23	1.10	0.95	0.86
		ε_t	-324.7	-324.7	-324.7	-324.7	-396.2	-396.2	-396.2
		ε_c	4865.5	3178.5	2218.8	1418.0	2562.3	1387.9	863.4
42%	61%	Deflection	2.00	1.77	1.62	1.47	1.36	1.15	1.03
		ε_t	-369.5	-369.5	-369.5	-369.5	-450.8	-450.8	-450.8
		ε_c	5891.3	3828.1	2664.4	1698.7	3080.1	1662.5	1032.6

Figures 5.2 reveals the pavement performances during flooding obtained using the HDM-4 and AASHTO 2008 models for the above-mentioned seven road groups. It is worth mentioning that the HDM-4 model has been calibrated for Queensland as the TMR-QLD uses a PMS tool based on HDM-III and reasonable factors were used for the AASHTO 2008 roughness prediction models. In the analysis, pavement performances were represented with ΔIRI in year 1 due to Mr losses of granular and subgrade layers. All these figures have been shown using the subgrade Mr loss (ΔIRI vs. subgrade Mr loss) for simplicity, although the analysis has considered both the subgrade and granular layers' Mr losses obtained from the EICM model.

Figure 5.2a shows the ΔIRI due to Mr loss at subgrade and granular layers for the Flexible pavement with Low Traffic loading and Poor strength road group (F_LT_P). The HDM-4 analysis reveals a constant ΔIRI at varying Mr loss. As initial pavement strength was very poor, the HDM-4 model predicted the highest ΔIRI at 0% Mr loss. As a result, the model provided the same value even at the higher Mr losses. However, the AASHTO 2008 model properly predicted the ΔIRI at varying Mr loss, which also showed an exponential increase in roughness after 30% Mr loss at subgrade layer and about 22% Mr loss at granular layers (see *Table 5.2*).

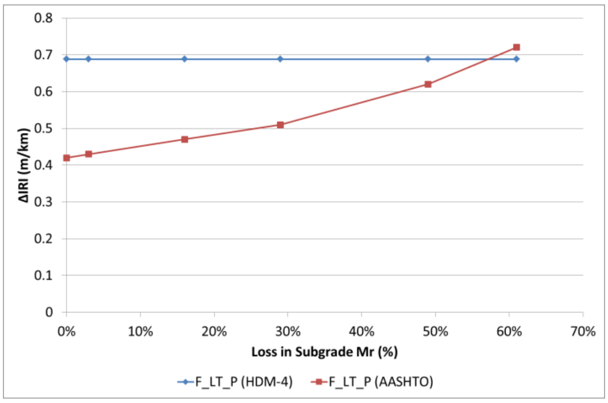
Similar to the above road group, the HDM-4 shows a constant value of ΔIRI for the Flexible pavement with Low Traffic loading and Fair strength road group (F_LT_F) after 20% of subgrade Mr loss (see *Figure 5.2b*), as pavement strength drops below a certain minimum value assumed in the HDM-4. Therefore, the HDM-4 model could not properly predict pavement performances at different Mr losses for the low traffic loading roads. A further analysis has investigated the reasons for over prediction with HDM-4, which is discussed in the next Section. In contrast, the AASHTO 2008 incorporated the change in roughness due to Mr loss for the low traffic loading roads.

Both the HDM-4 and AASHTO 2008 showed similar trends for the medium traffic loading road groups, i.e., Flexible pavement with Medium Traffic loading and Fair strength road group (F_MT_F) and Flexible pavement with Medium Traffic loading and Strong strength road group (F_MT_S) (see *Figures 5.2c and 5.2d*). It was observed that the HDM-4 model provided higher ΔIRI at each Mr loss in year 1, and there was almost a constant gap of ΔIRI values between the HDM-4 and AASHTO methods for each Mr loss. Although both of these roughness prediction models were based on different factors, they showed similar trends for the medium traffic loading, i.e., higher ΔIRI at lower granular and subgrade Mr values. It indicates that a pavement performs poorer if it gets weaker.

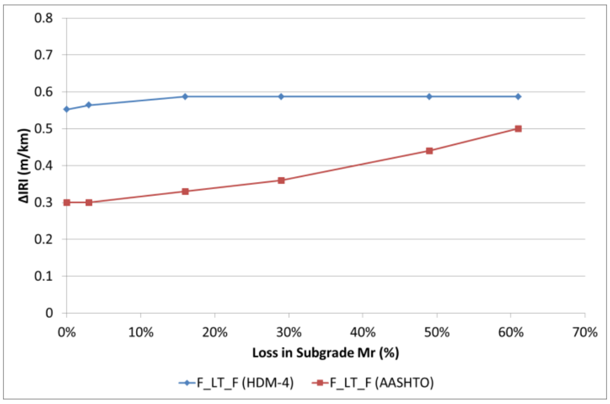
Figures 5.2e, 5.2f, and 5.2g revealed that both HDM-4 and AASHTO 2008 provided very close results with similar trends of ΔIRI change at varying Mr values for the heavy loading road groups. As expected, these results show that the stronger (or thicker) a pavement the better is its performance. It can be concluded that both of these models are suitable for predicting ΔIRI in year 1 to assess impact of Mr loss due to extreme moisture intrusion. A further analysis showed that an appropriate assumption of SN is vital for ΔIRI estimation with the HDM-4 model.

Figure 5.3a revealed pavement performances using the AASHTO 2008 model at varying Mr values for different high traffic loading groups. It shows that at the same type of loading, the poor pavement performs the poorest, then the fair one followed by the stronger road group. In all the cases higher rates of roughness increase have been observed beyond the 30% of subgrade Mr loss and 22% of granular Mr loss values. Similarly, *Figure 5.3b* shows the similar trend of ΔIRI due to Mr loss for fair strength road groups. It revealed that at the same strength level, the lower traffic loading road group performs the poorest, then moderate traffic loading road group, and the high traffic road group performs the best. The major reason is related to their set standards shown in *Table 5.5*. As the high traffic loading road groups have better standards, they are well maintained and perform well. On the other hand, the low traffic loading road groups have relaxed standards because of low loading, and get less attention for maintenance. Therefore, at the same strength level, a low traffic loading road group performs poorly. However, if the standards are the same for all these road groups, the low traffic loading roads perform the best followed by the medium traffic roads.

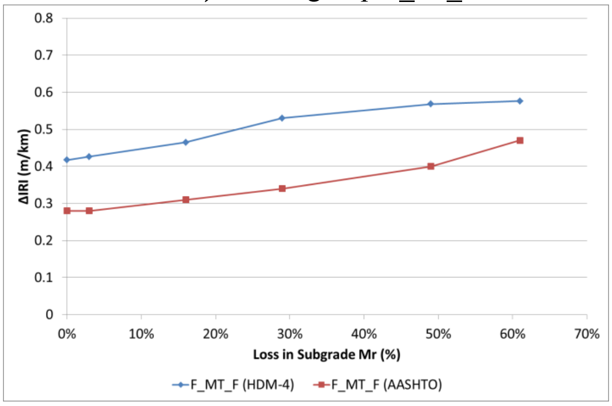
In all the cases, higher rates of roughness increase were found after the 30% of subgrade Mr loss and 22% of granular Mr loss, i.e., after +10% of ($S\text{-}S_{opt}$). Similar to *Table 5.5*, both the *Figures 5.3a and 5.3b* reveal that the stronger and high traffic loading road groups perform well during extreme moisture intrusion.



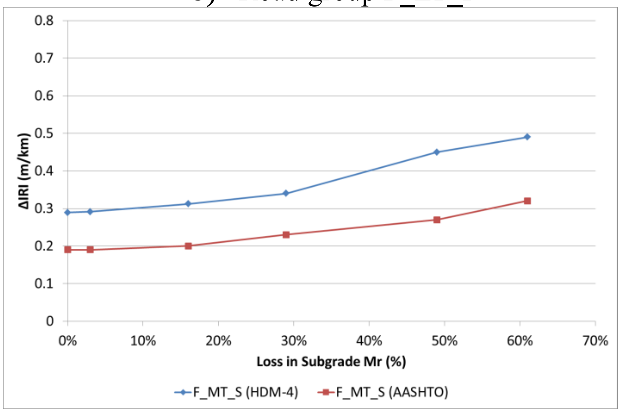
a) Road group F_LT_P



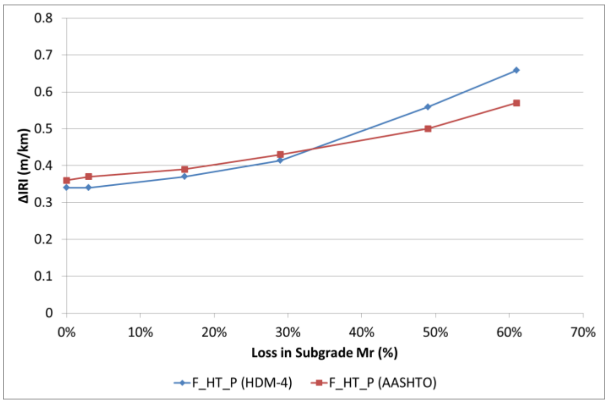
b) Road group F_LT_F



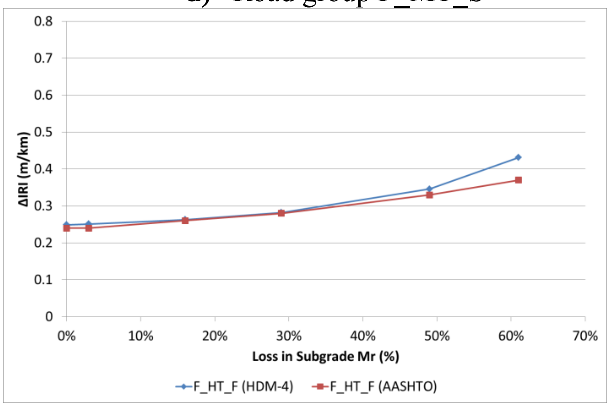
c) Road group F_MT_F



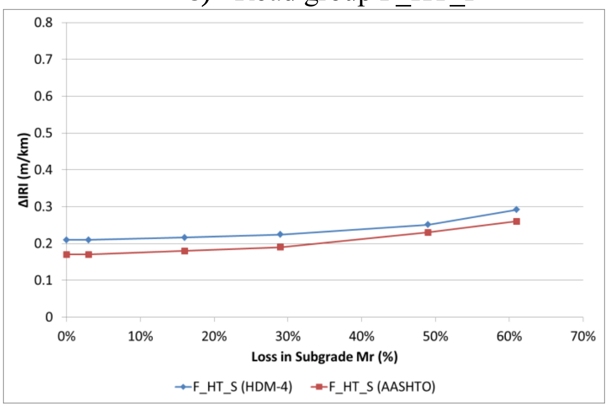
d) Road group F_MT_S



e) Road group F_HT_P

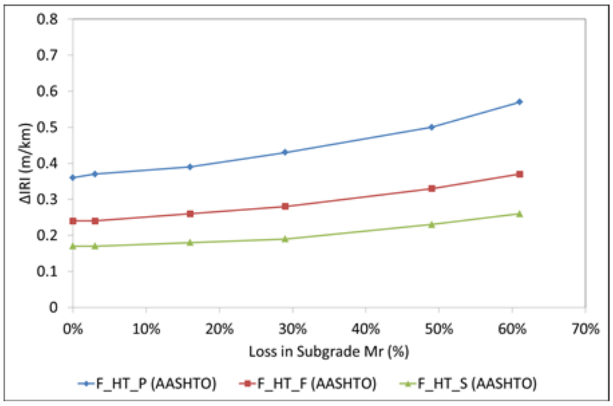


f) Road group F_HT_F

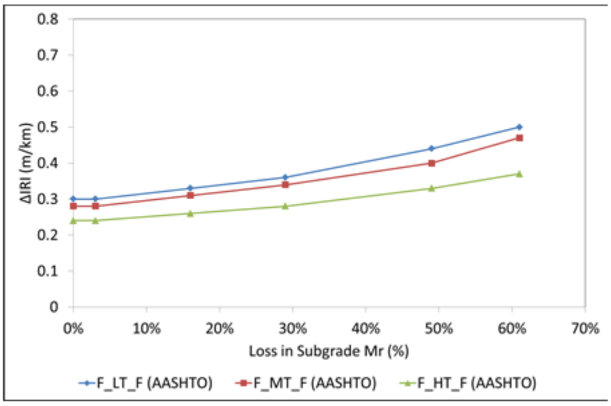


g) Road group F_HT_S

Figure 5.2: ΔIRI derived from the HDM-4 and AASHTO 2008 models due to Mr loss at granular and subgrade layers



a) for the high traffic loading groups



b) for the fair strength road groups

Figure 5.3: ΔIRI due to Mr loss at granular and subgrade layers using the AASHTO 2008

Table 5.5: Derivation of flood-resistance among the road groups

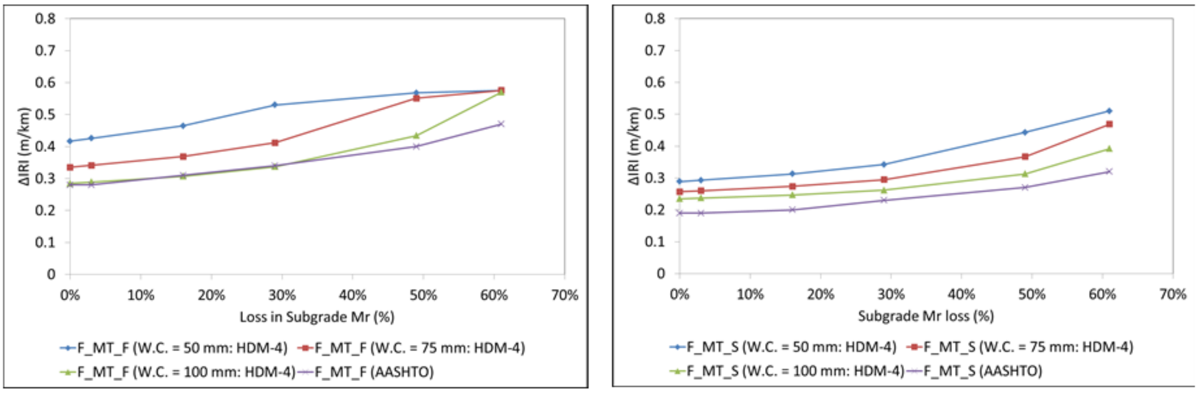
Road Group	Set Optimum Maintenance Standard	Change in ΔIRI/change in Mr loss of subgrade (obtained from *Figure 5.2*)
F_LT_P	5.5 IRI	0.52 (0.31/60%)
F_LT_F	5.0 IRI	0.33 (0.20/60%)
F_MT_F	4.0 IRI	0.32 (0.19/60%)
F_MT_S	4.0 IRI	0.22 (0.13/60%)
F_HT_P	4.5 IRI	0.35 (0.21/60%)
F_HT_F	4.0 IRI	0.23 (0.14/60%)
F_HT_S	4.0 IRI	0.15 (0.09/60%)

5.3 Matching Results of the HDM-4 and AASHTO 2008

The above HDM-4 and AASHTO 2008's roughness prediction due to extreme moisture intrusion or a flood revealed that both the results were close for heavy traffic loading cases only. It was noticed that HDM-4 roughness progression model is related to several factors, i.e., pavement strength, age, initial roughness, new and old surfacing thicknesses, cracking, potholing, rutting, traffic loading and environment (Odoki and Kerali, 2000). It was observed in the current research that ΔIRI is very sensitive to SN which is linked to subgrade Mr. Moreover, pavement different layers' thicknesses are also linked to the SN. In the current analysis, SN and different granular layers thicknesses were assumed because of not having detailed data in the database.

The assumed asphalt surface layer thicknesses were high, i.e., 100 mm, for the heavy loading road groups, which could increase their initial SNs (see *Table 5.1*). As a result, it provided lower ΔIRI for these roads which matched closely with the AASHTO 2008 results. On the other hand, surface layer thicknesses have been assumed as 50 mm for the low and medium traffic loading road groups (see *Table 5.1*). In view of that, further analysis were carried out by increasing surface layer thickness from 50 mm to 75 and 100 mm or initial SN values to the medium traffic loading road groups to check their impact on ΔIRI at different Mr loss.

Figure 5.4a shows that if the SN was assumed higher with 100 mm wearing course for the F_MT_F road group, then the HDM-4 roughness progression results would match with the AASHTO 2008. However, the road deterioration would be at higher rate for the HDM-4 case after 30% of loss in subgrade Mr and around 22% of loss in granular Mr (see *Table 5.2*). It also shows that a pavement with higher surface thickness (higher initial SN) performs better. Similarly, *Figure 5.4b,* generated for the F_MT_S, provided the same. It can be said that ΔIRI values for the pavement with 100 mm asphalt layer (highest initial SN) are close to the results derived from the AASHTO 2008.



a) for the F_MT_F b) for the F_MT_S

Figure 5.4: Comparison of ΔIRI due to Mr loss at granular and subgrade layers with AASHTO 2008 and with different surface layer thicknesses (or initial SN values) with HDM-4 for the F_MT_F and F_MT_S

5.4 Validation of Deterioration Prediction

The ΔIRI in year 1 obtained using the AASHTO 2008 and HDM-4 was checked with the real data that were possible to capture after the January 2011 flood in Logan, especially for the Mount Lindesay highway and Waterford-Tamborine road data.

It was observed that the higher the probability of a flood the higher is its impact on a pavement. Therefore, this analysis assumed that Mr loss would be the highest at the 100% probability of flooding. It was found that a 100% probability of flood means about 75% of ($S-S_{opt}$), which will be discussed later on. Therefore, a 20% probability of flooding, which was found for the January 2011 flood in Logan, may be considered as around 15% of ($S-S_{opt}$) or 40% subgrade Mr loss or 28% granular layer' Mr loss (see *Table 5.2*).

Table 5.6 provides the predicted and actual results comparison for these two roads. The predicted ΔIRI in year 1 for the Mount Lindesay Highway were found 0.23 IRI, 0.25 IRI and 0.27 IRI respectively using the new RD models, AASHTO 2008 and HDM-4; which were close to the actual data of 0.26 IRI. Similarly, the predicted ΔIRI in year 1 for the Waterford-Tamborine road were found 0.36 IRI and 0.38 IRI respectively using the AASHTO 2008 and HDM-4. The actual data provided only one value after 5 months which was 0.20 IRI. Assuming first six-month' deterioration rate was the same for year 1; the predicted ΔIRI in six months would be 0.18 IRI and 0.19 IRI respectively with the AASHTO 2008 and HDM-4. Therefore, these predicted deterioration results matched quite closely with the actual data.

A t-test was also undertaken between the predicted ΔIRI and actual data, which also showed that the difference was statistically insignificant (see *Table 5.6*). As a result, it is concluded that the RD models are sound, as they provided consistent results to the AASHTO 2008 and HDM-4 roughness models and actual data.

Table 5.6: Validation of the predicted results with actual data and t-tests for two major roads in Logan

Road Group	Road Name	Roughness vs. Time Data (at 20% Probability of Flooding)				Unpaired t-test Results Among the AASHTO 2008 and HDM-4 Findings			
		Actual Data	RD Model Results	AASHTO 2008 Results (from *Figure 5.4*)	HDM-4 Results (from *Figure 5.4*)	Two Tailed P value	Mean Difference Within 95% Confidence Interval	t value	Remarks on the Data Difference
F_MT_S	Mount Lindesay Highway	Jan 11: 3.05 IRI April 11: 3.08 IRI Feb 12: 3.31 IRI	Jan 11: 3.05 IRI April 11: 3.10 IRI Feb 12: 3.28 IRI	Jan 11: 3.05 IRI ΔIRI in Yr-1: 0.25 IRI Jan 12: 3.30 IRI	Jan 11: 3.05 IRI ΔIRI in Yr-1: 0.27 IRI Jan 12: 3.32 IRI	0.9616 Df: 4	Yes, within the range	0.0544	Not statistically significant
F_MT_F	Waterford-Tamborine Road	Jan 11: 3.01 IRI June 11: 3.21IRI	No RD model because of incorrect data	Jan 11: 3.01 IRI ΔIRI in Yr-1: 0.36 IRI Jan 12: 3.37 IRI	Jan 11: 3.01 IRI ΔIRI in Yr-1: 0.38 IRI Jan 12: 3.39 IRI	0.9730 Df: 4	Yes, within the range	0.0382	Not statistically significant

5.5 Finding a Better Flood-resistant Pavement

Detailed analyses in obtaining flood-resilient pavements are shown in the next chapter. Some key results for the flexible pavements are discussed here. The relationship of ΔIRI in year 1 at varying Mr loss of subgrade given in *Figure 5.2* was used to get the pavements that perform well during extreme moisture intrusion. The results are shown in *Table 5.5*. A deterioration rate due to subgrade Mr loss using change in ΔIRI in year 1/change in Mr loss of subgrade, termed as ΔIRI/MrL, gives performance results of these roads during a flood or an extreme moisture intrusion. It reveals that Flexible pavement with High Traffic loading and Strong strength road group (F_HT_S) has the lowest deterioration rate at varying subgrade Mr loss. This is a stronger pavement road group and has better maintenance standard of 4.0 IRI with high traffic loading; hence, it gets appropriate attention. Therefore, it performed the best as a flood-resistant road; then come F_MT_S and Flexible pavement with High Traffic loading and Fair strength road group (F_HT_F). After detailed investigation, it was identified that a rigid pavement generally performs better than a flexible and a composite road; which is shown in the following chapter. In fact, a rigid and strong pavement with a high standard road would perform the most satisfactorily.

It is worth noting that the analysis uses pavement types, traffic loading and pavement strength as key variables, and hence flood-resilient pavements results are obtained based on these factors. Other impacting factors such as drainage conditions are assumed to be similar for the road groups.

5.6 Obtaining the Change in (S-Sopt) During an Extreme Moisture Intrusion

As mentioned earlier, the RD models provide ΔIRI in year 1 at different probabilities of flooding. The proposed ΔIRI values in year 1 at 100% probability of flooding along with the ΔIRI vs. Mr loss relationships shown in *Figure 5.2* were used for the seven flexible pavement road groups to obtain i) corresponding Mr loss at granular and subgrade layers, and ii) % change in ($S\text{-}S_{opt}$). The results give change in ($S\text{-}S_{opt}$) after a flood event. The realistic values of ΔIRI in year 1 at 100% probability of flooding and Mr losses were considered which ultimately provided results for only two road groups (see *Table 5.7*). Major reasons for the unrealistic ΔIRI in year 1 could be due to wrong data and/or not having any after flood data to generate a flood TPM. The analysis results reveal that change in ($S\text{-}S_{opt}$) during a flood may be between 45 and 60%. The percentage change in ($S-S_{opt}$) vs. Mr loss given in *Table 5.2* shows that the maximum percentage in ($S-S_{opt}$) may be up to around 75%. Therefore, it has been concluded that change in ($S-S_{opt}$) could vary in the range of 45-75% after a flood; whereas, it can go up to 25-30% at normal condition.

The major limitation of this result is that it was not derived from any geotechnical investigations nor validated with field data. Therefore, these are indicative results, but they matched closely with actual site data. As a result, these models also confirmed the new RD models with flooding.

Table 5.7: Derivation of S-Sopt after a flood event

Road Group	ΔIRI in year 1 at 100% probability of flooding	Mr loss in Subgrade (%) (obtained from *Figure 5.2*)	Mr/Mr_{opt} for Subgrade	Mr loss in Granular Layers (%) (obtained using relationship given in *Table 5.2*)	Mr/Mr_{opt} for Granular Layers	% Change in ($S-S_{opt}$)
F_LT_P	0.85	76%	0.24	55%	0.45	47%
F_HT_P	0.75	83%	0.17	63%	0.37	58%

5.7 Pavement Life-cycle Performances at Varying Mr Loss

A pavement life-cycle analysis by varying Mr loss in year 1 has been carried out to assess the impact of Mr loss on its performance. The HDM-4 model was used for this purpose. As the Flexible pavement with High Traffic loading and Poor strength road group (F_HT_P) has Mr loss values during flooding (see *Table 5.7*); hence, this road group was chosen in the analysis. The results revealed that this road group performed the best at 0% Mr loss scenario and the worst at 63% Mr loss at granular layers during a flood or extreme moisture intrusion. The higher the Mr loss in year 1 the poorer was the life-cycle performance or deterioration rate. If 63% of Mr was lost at granular layers in year 1, then three 30 mm overlay was necessary in life-cycle; on the other hand, 0%, 21% and 34% Mr loss scenarios required two 30mm overlay. In general, a 30 mm overlay was needed in every 6, 8 and 10 years for the 63%, 21/34% and 0% Mr losses in year 1 respectively (see *Figure 5.5*). The economic results show that the Net Present Value (NPV) at 0% and 63% Mr loss at granular layers were almost the same; whereas, the NPV/Cost was 25.80 for 0% Mr loss case and 18.54 for 63% Mr loss at granular layers case. This road group requires one additional overlay of 30 mm if a flood or an extreme moisture intrusion occurred in year 1 which has lowered economic benefits per A$ investment.

Generally, stabilisation is a good option for rehabilitation. In developing the optimum strategy at normal conditions, the six treatments practiced by TMR_QLD was used. As a result, stabilisation was not considered in this specific analysis. However, it was used along with different alternatives in the HDM-4 analysis to get the proposed pre- and post-flood maintenance strategies.

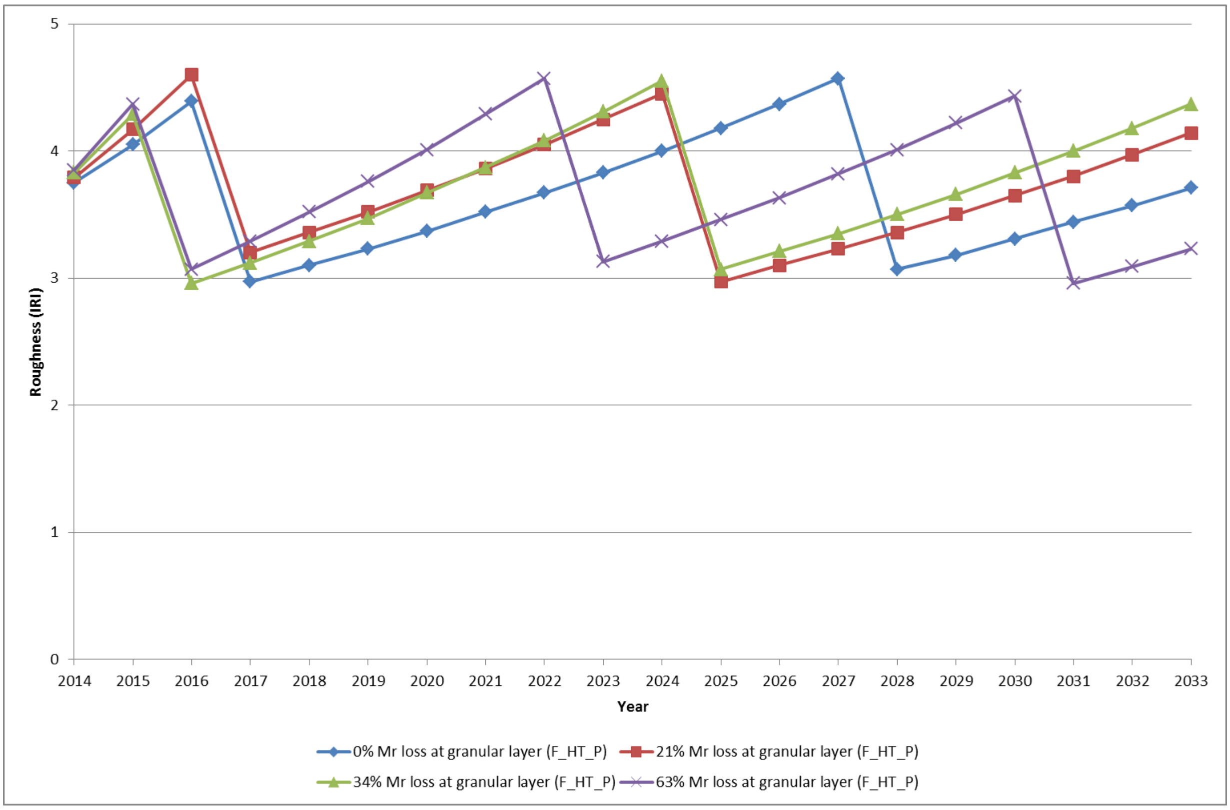


Figure 5.5: Pavement life-cycle performance due to Mr loss in year 1 for the F_HT_P

5.8 Discussions on the Main Findings

Previous studies reveal that an Mr loss due to moisture intrusion influences pavement design and performances. Generally, heavy or localised flooding, heavy rainfall with water ponding and poor sub-surface drainage may increase moisture in pavement layers. The review of literature has not found any study that can address pavement performances during moisture intrusion in granular and subgrade layers. Therefore, the current study has aimed at investigating the impact of change in Mr due to moisture on pavement performances. The

HDM-4 and AASHTO 2008's roughness prediction models have been used to estimate ΔIRI change in year 1 due to Mr loss at granular and subgrade layers. These results helped verifying the new RD models.

At first, reasonable material properties, applicable to Queensland, were used and assumed if needed. The EICM model of AASHTO 2008 has been utilised to obtain the Mr values at different pavement layers due to moisture intrusion. The different Mr values could give SN values, which were used as inputs to the HDM-4 model. Similarly, the MET was used to get deflection, ε_c and ε_t at varying Mr for all defined road groups, which were used as inputs to the AASHTO 2008 roughness prediction models. The study has considered seven selected flexible pavement road groups in Queensland that covered about 50% of the network. The roads were grouped considering different traffic loading and pavement existing strength to assess impact of these factors.

In general, pavement SN losses are higher when Mr losses are also higher. Higher deflection, ε_c and ε_t values, have been found when Mr of granular and subgrade layers were decreased, hence a pavement becomes weaker at that condition. The observed Mr loss values at different ($S-S_{opt}$) were reviewed with the previous studies and found to be reasonable.

The comparison results of ΔIRI for the low traffic loading road groups revealed that the AASHTO 2008 model well predicted pavement performances due to Mr loss. However, the HDM-4 did not produce any sound results because of assumed poor initial SN values. The AASHTO 2008 model showed a higher rate in increase of roughness after 30% Mr loss at subgrade layer and about 22% Mr loss at granular layers. Both the models showed similar trends for the medium traffic loading road groups. It was observed that HDM-4 provided higher ΔIRI in year 1 at each Mr loss, and there was almost a constant gap of ΔIRI values between the HDM-4 and AASHTO results. Similar to the low traffic loading road groups, assumption of initial poor SN values affected the results of the HDM-4 model.

On the other hand, better matches were observed between the HDM-4 and AASHTO 2008 results for the heavy loading road groups. The asphalt layer thicknesses on these roads were assumed higher compared to other groups, which gave higher initial SN values to predict roughness in the HDM-4 analysis. As a result, both the HDM-4 and AASHTO 2008 provided very close results with similar trends of ΔIRI change at varying Mr values.

It was observed that at the same loading level, the weakest pavement performs the poorest, followed by the fair one. The strongest road group performs the best. Similarly, at the same strength level, the lower traffic loading road group performs the poorest, followed by the medium traffic loading road group and the high traffic road group. This is valid when the high traffic loading road groups have higher standards for maintenance. It was observed that if standards were the same, then the low traffic loading road groups performed the best.

As the ∆IRI in year 1 results obtained from HDM-4 for the low and medium traffic loading were found higher compare to the AASHTO 2008 model due to lower asphalt surface thickness, another assessment has been done by increasing asphalt surface layer thicknesses (or initial SN values) for the medium traffic loading road groups. Interestingly, the results have shown that if the SN value was assumed higher (or appropriately), then the HDM-4 roughness progression results would match with the AASHTO 2008. It also showed that a pavement with higher surface layer (higher initial SN) performed better. In all cases, it showed a higher rate of road deterioration for the HDM-4 after 30% loss in subgrade Mr and 22% loss in granular Mr.

It was mentioned earlier that Mr values closely matched the previous studies. Using these Mr loss values as inputs, the predicted ∆IRI in year 1 was verified with the actual data for two major roads in Logan. Therefore, these results on ∆IRI in year 1 at varying Mr loss deemed acceptable for further analysis. In addition, the RD models were compared with these predicted ∆IRI, and found a close match. A t-test revealed that all these results are acceptable.

The indicator of ∆IRI/MrL for seven road groups was used to get pavement performances during an extreme moisture intrusion or a flood. A detailed analysis was undertaken to obtain the flood-resilient pavements, which will be discussed in the next chapter. The current results showed this for the flexible pavements only. It showed that the F_HT_S road group performed the best as it has the lowest deterioration rate due to subgrade Mr loss. After that the F_MT_S and F_HT_F performed well.

Another investigation has been done to obtain maximum change in (S-S_{opt}) during an extreme moisture condition. Therefore, ∆IRI in year 1 at different probabilities of flooding were used to obtain corresponding Mr loss at granular and subgrade layers, and ultimately change in (S-

S_{opt}). It was observed that change in (S-S_{opt}) could vary between 45 and 75% after a flood event; whereas, it can go up to 25-30% at normal condition.

The HDM-4 model has been used to assess the impact of Mr loss in year 1 on pavement life-cycle performances, and the F_HT_P was used as an example. As expected, this road group performed well at 0% Mr loss (or normal condition) and the worst at flooding scenario or extreme moisture condition when 63% Mr was lost at granular layers. The flooding condition in year 1 needs an additional overlay of 30 mm, which comes 4 years earlier than the normal condition. As a result, economic benefits were lower per investment.

Pavement performance data with flooding is very rare, and a field investigation was outside the scope. Both the AASHTO and HDM-4 are well utilized models and can predict pavement performances. The HDM-4 model has been calibrated locally. The AASHTO model provided indicative results as some of its factors were extracted from the USA studies and some are valid for local conditions. However, the relative comparisons of models are highlighted when a flood is considered. Furthermore, the results were checked at different stages for reliability. Although, the findings were indicative, they matched closely with some point data from the field. Ullidtz (1998) tried to compare pavement performances using HDM-III and AASHTO models. This study is an extension of that effort, and it also considered flooding in measuring pavement performances.

5.9 Summary

This chapter has shown pavement performances after a flood due to Mr loss at granular and subgrade layers. The AASHTO 2008 and HDM-4 roughness models were used to get the results. One of the major aims of this research was to check consistency of the new RD models with these results and actual observations. Although a few roads' data were possible to compare, all the findings matched closely; and the RD models were verified. The chapter discussed obtaining key variables of the two models, matching of their results, validation of the RD models, obtaining flood resilient pavements using the new indicator (ΔIRI/MrL) data, change in (S-S_{opt}) because of a flood and pavement life-cycle performances due to Mr loss. Detailed discussions on the major findings are included.

CHAPTER SIX: OBTAINING FLOOD-RESILIENT PAVEMENTS[3]

This chapter discusses attaining a flood-resilient pavement. Three approaches were used for this purpose: i) use of the new ΔIRI/Pr values, ii) comparing the performances of different pavement types under flooding, and iii) use of the new ΔIRI/MrL values (*Figure 3.6*). All these provided results on sound flood-resilient pavements, which are discussed below. *Figure 3.1* shows the components highlighted in this chapter.

6.1 Pavement Performances at Different Probabilities of Flooding

The new RD models were used to get the new ΔIRI/Pr values. As the RD models were derived at different probabilities of flooding, these ΔIRI/Pr values were used to quantify pavement performances with flooding.

The new IRI and rutting-based RD models for flexible and composite pavements and IRI-based RD models for rigid pavements have been derived for all the road groups. The TPMs with flooding and under normal conditions were generated for a road group using 10 to 12 years of data, which were used in the Monte Carlo simulation for RD modelling. Detailed have been discussed in Chapter Four. As an example, a specific road group of Flexible pavement with High Traffic loading and Strong strength (F_HT_S) has been chosen here, which IRI and rutting-based RD models at different probabilities of flooding are shown (see *Figure 6.1*). In the simulation process, i) road condition at the start was considered as excellent, and ii) the RD models were considered valid up to 2 to 3 years.

Figure 6.1 shows how poor a pavement performs with a change in the probability of flooding. The rutting-based RD model showed a more dispersed trend in this case. The result revealed that ΔIRI at 100% probability of flooding in year 1 was 0.62, while it was 0.35 at no flooding. Therefore, ΔIRI increases due to a certain flood was 0.27; which value was adequate to change in post-flood rehabilitation treatment selection for a high traffic loading road group like F_HT_S. In addition, the impact of a flood on a pavement performance also relates to a maintenance standard. The ΔIRI increase for the F_HT_S was low as this road

[3] *The findings provided in this chapter has resulted a Journal Paper: http://ascelibrary.org/doi/abs/10.1061/JPEODX.0000007*

group has a high maintenance standard. These RD models showed the actual pavement deterioration trends with a flood.

After a critical assessment of the RD models, it was found that the IRI-based RD models were valid for 13 road groups and the rutting-based RD models for 7 road groups. No rutting models have been developed for the rigid pavements, as they do not have rutting. One road group (Composite pavement with High Traffic loading and Poor strength, C_HT_P) did not have any data, and three road groups (Flexible pavement with Medium Traffic loading and Poor strength: F_MT_P, Composite pavement with Low Traffic loading and Poor strength: C_LT_P and Rigid pavement with High Traffic loading and Poor strength: R_HT_P) did not have adequate data nor had IRI jumps to get flooding TPMs. Moreover, the IRI-based RD models for ten road groups and the rutting-based RD models for seven road groups were not appropriate due to i) having inconsistency when compared with other pavement types, loading and strengths; ii) providing highest ΔIRI/Δrutting at 0% probability of flooding or at normal conditions; and iii) having no change in ΔIRI/Δrutting at 0% probability of flooding or at normal conditions. In fact, (ii) and (iii) reveal an abnormality in road deterioration. These could be due to inconsistent data which affected the derived normal and flooding TPMs of a road group. These are discussed in Appendix 4C.

The current study has generated ΔIRI/Pr values from the RD models. The lowest gradient value indicates a better performance with a change in probability of flooding.

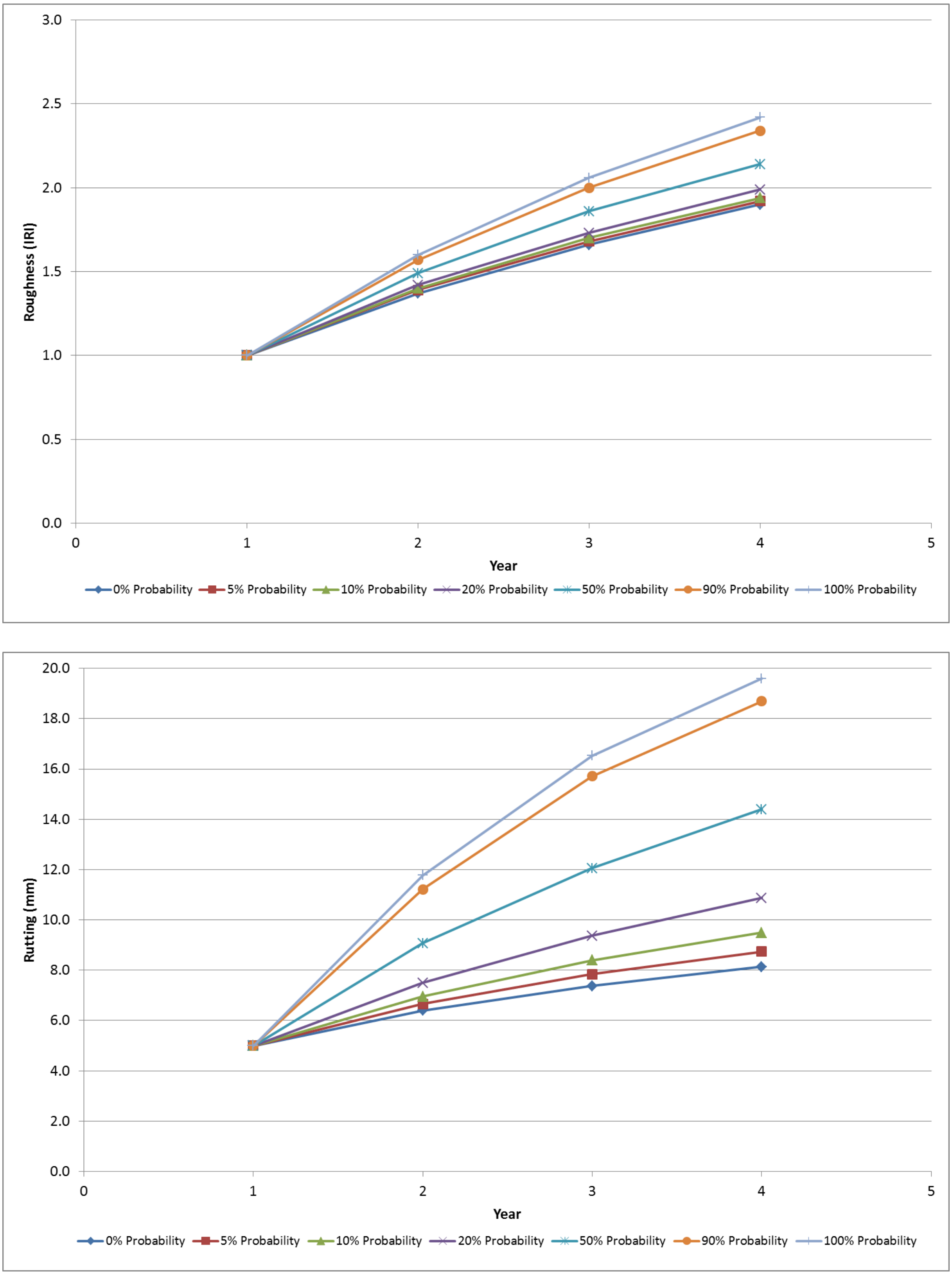


Figure 6.1: The new (a) IRI-based and (b) Rutting-based RD models for the F_HT_S

6.2 Impact of Different Types of Pavement, Loading and Strength on Pavement Performances with Flooding

Generally, a pavement performance with flooding for some initial years (up to 2 to 3 years) depends on pavement type, loading, pavement strength and set maintenance standards. Moreover, it has a linkage with a flooding probability; as a highest probability results in the poorest performance.

Considering pavement type, a rigid pavement performs better than composite and flexible road groups incorporating flooding. Both composite and flexible road groups show similar performances up to 2 to 3 years. The stabilised layer of a composite pavement becomes granular after some years; hence composite and flexible pavements behave in the same way. As an example, *Figure 6.2* shows a comparison of pavement performance with flooding for three different types of road groups, that is, F_HT_S, Composite pavement with High Traffic loading and Strong strength (C_HT_S) and Rigid pavement with High Traffic loading and Strong strength (R_HT_S). *Figure 6.2* shows that a rigid pavement (R_HT_S) performed the best at any probability of flooding, and flooding impact was not critical for this road group. This is also reported by Gaspard, et al. (2006). The current results show that a flexible pavement performed better than a composite one at the initial years after flooding; although Gaspard, et al. (2006) observed a better performance for a composite pavement than a flexible one. As a result, it is settled that a rigid pavement is more flood resilient.

A critical review of the RD models showed that the high loading road groups performed better than the medium loading and low loading roads; as generally, the high loading roads have higher maintenance standards. However, if their maintenance standards are the same then the low loading road groups perform the best. In general, a higher standard indicates a better pavement maintenance practice at a set lower IRI. *Figure 6.3* reveals an example of pavement performances with flooding for three loading scenarios. Three road groups has been chosen here, that is, F_HT_S, Flexible pavement with Medium Traffic loading and Strong strength (F_MT_S) and Flexible pavement with Low Traffic loading and Strong strength (F_LT_S). The results show that the low loading road group with the lowest standard of 4.5 IRI performed the worst with flooding. Both the medium and high loading road groups had the same set standards of 4.0 IRI. Hence, the medium loading road group performed better with flooding during the initial years than the high loading road group.

The impact of loading on pavement performances with flooding generally means that a road performs better if it carries low loads. It is worth noting that the simulation analysis considered road condition or TPMs, and no impact of loading during the flooding period was analysed. The current results showed that the maintenance standard, which relates to loading, influences pavement performances. It was observed that a higher standard road with lower loading can ensure better performance with a flood.

Another key observation is that a strong pavement road group performs the best; and a fair pavement performs better than a poor pavement. Moreover, it is known that a strong pavement road group has a higher standard. *Figure 6.*4 shows an example of pavement performances with flooding for two different strength road groups. The outcomes did not provide results for the three pavement strength types to compare. In fact, Flexible pavement with High Traffic loading and Fair strength (F_HT_F) road group did not have a logical RD model because of inconsistent data. Therefore, a comparison has been done among F_HT_S and Flexible pavement with High Traffic loading and Poor strength (F_HT_P). As expected, the strong pavement road group performed much better in its service life with flooding than the poor one, which is supported by Gaspard, et al. (2006). This indicates that a strong pavement is more flood-resilient.

In addition, it was noticed that the high standard road groups performed better than moderate and in turn low standard roads. High standard roads are strong (with higher pavement thickness) and are maintained efficiently as they carry heavy traffic. The TMR-QLD data revealed that these roads have frequent maintenance done. As a result, a high standard road has a lower deterioration rate during a flood.

Roughness jumps were observed from year 2 to 3 for the F_LT_S (see *Figure 6.3*) and F_HT_P road groups (see *Figure 6.4*). Simulation results were based on the TPMs derived from the IRI vs. time data for the respective road groups, and these results were generally acceptable. Therefore, it was difficult to know why a different trend was found for the two cases. However, these did not have any impact on the results while assessing pavement performances with flooding at different types, loading and strength.

The above findings suggest that road authorities may consider pavements flood resilience before a flood occurs by providing strong and rigid pavements in vulnerable locations. This can be done through an appropriate rehabilitation with strengthening overlay and/or stabilising granular layers and replacing with a rigid pavement. Recently, the World Bank (2013) planned to invest in a climate-resilient road project through rehabilitation of 1,000 km of paved and unpaved roads in Mozambique. They mainly concentrated on stabilisation of granular layers based on previous experiences only. Similar approach has been considered for rehabilitation of flood damaged roads in Queensland (Fisher and Smith, 2014).

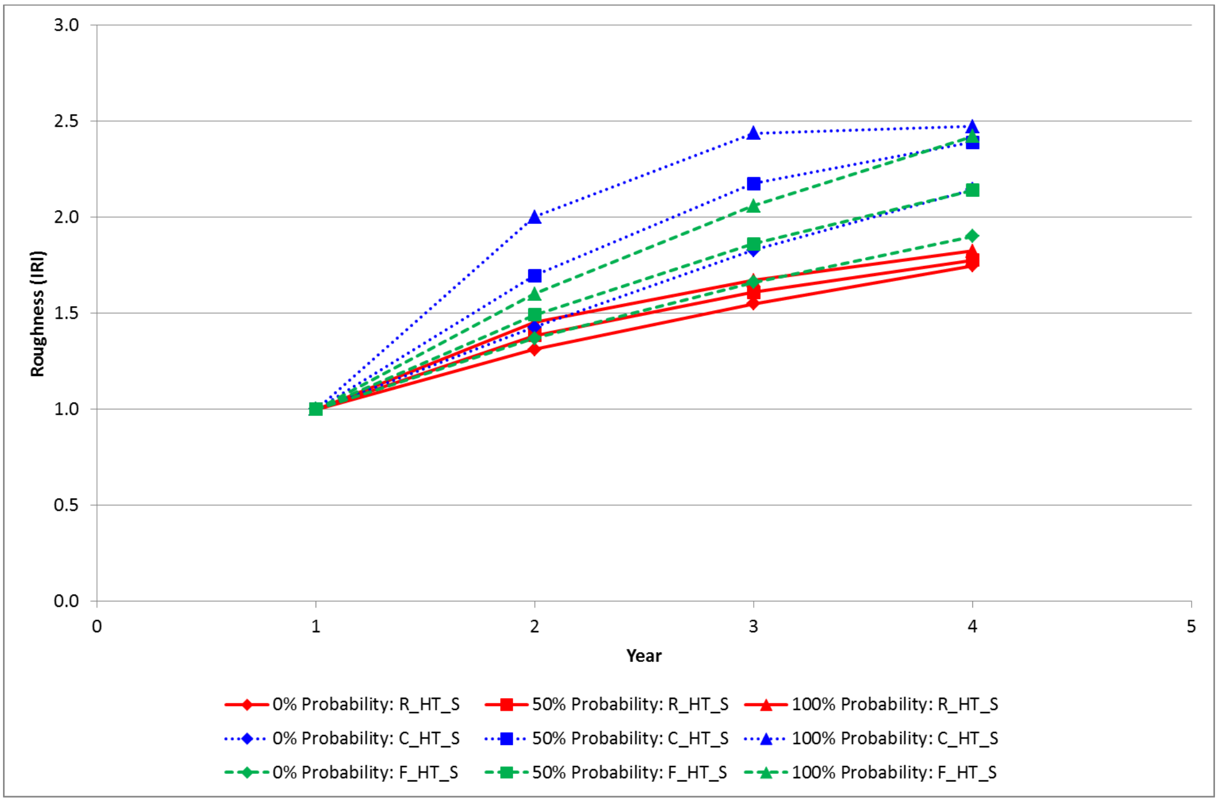


Figure 6.2: Comparison of pavement performances with flooding using three types of pavement

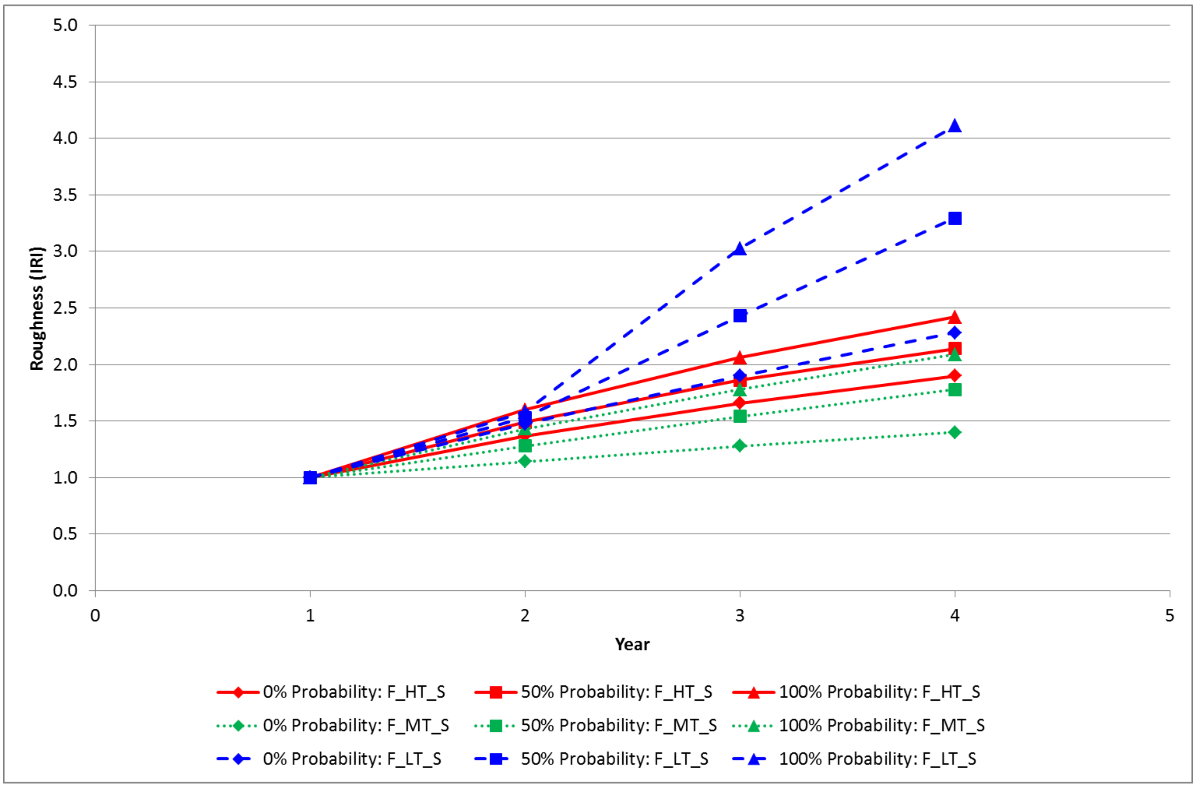


Figure 6.3: Comparison of pavement performances with flooding using three types of traffic loading

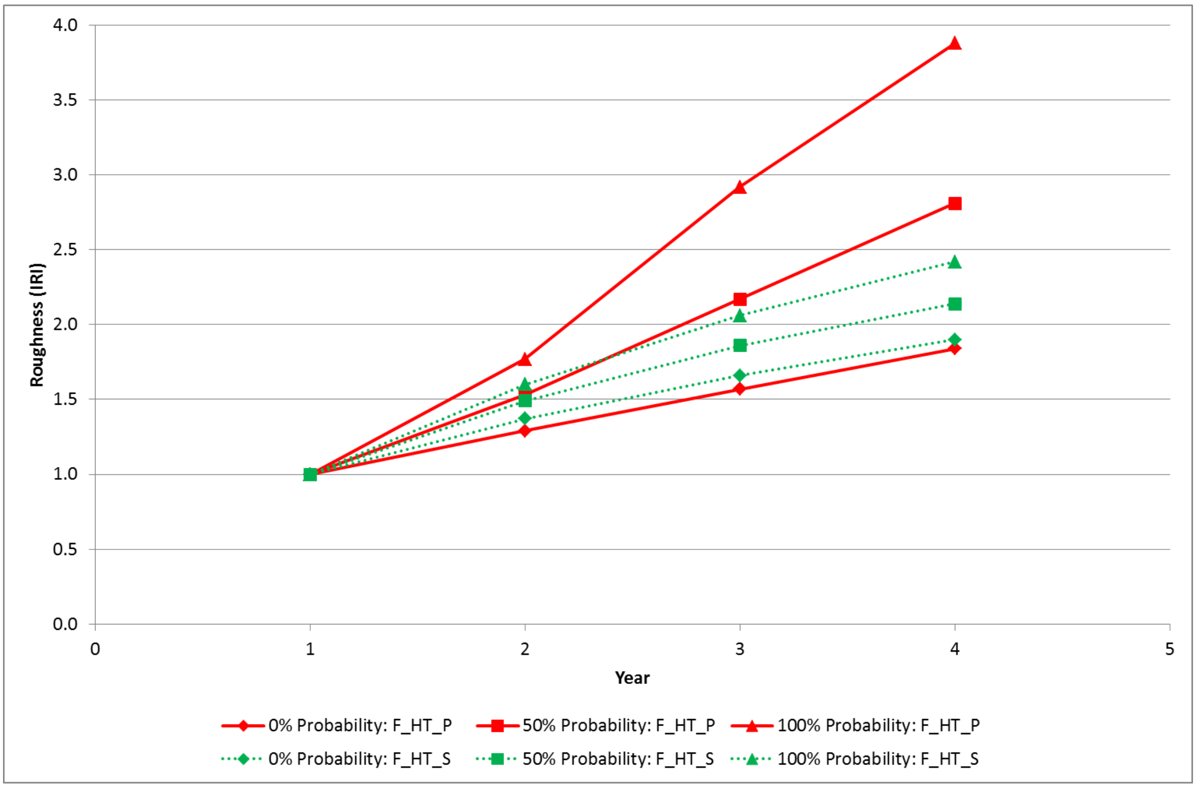


Figure 6.4: Comparison of pavement performances with flooding using two types of strength

6.3 Impact of Mr Loss on Pavement Performances During Flooding

The results obtained from the seven flexible pavement road groups provide ΔIRI vs. Mr loss relationships in year 1 due to flooding. It was observed that the higher the Mr loss, the poorer the pavement performances or the higher the ΔIRI. An exponential rate was observed for ΔIRI after 30% of subgrade and 22% of granular layers' Mr losses. In other sense, a higher rate was found when the degree of saturation – optimum degree of saturation ≥ 10%.

Similar to the RD model results, the current study has generated a new ΔIRI/MrL for these road groups. The lowest gradient indicates a better performance with a change in Mr loss. *Figure 6.5* shows that the F_HT_S had the lowest value, 0.15, for ΔIRI/MrL during flooding. Therefore, it performed the best during flooding. As the F_HT_S is a strong road group with a higher maintenance standard and is maintained appropriately because of high loads, it performed the best. Apart from this road group, the F_MT_S and F_HT_F also performed well due to their higher standards, and their gradient values were found 0.22 and 0.23, respectively. The F_MT_S is a strong road group with a higher standard, and hence it performed the second best.

At the same strength group, a high loading road performs the best as it has the highest standard and receives appropriate maintenance. On the other hand, within the same loading group, a strong pavement performs better than a fair pavement, and a poor pavement performs the worst. All these provide consistent results on the flood-resilient pavements obtained in the previous Section. Therefore, it has been concluded that a flexible flood-resilient pavement has to be strong with higher standard. A road may be converted to a strong one through strengthening overlay and/or granular and subgrade layers stabilization. In addition, a pavement can perform well with flooding if it is transformed into a rigid pavement.

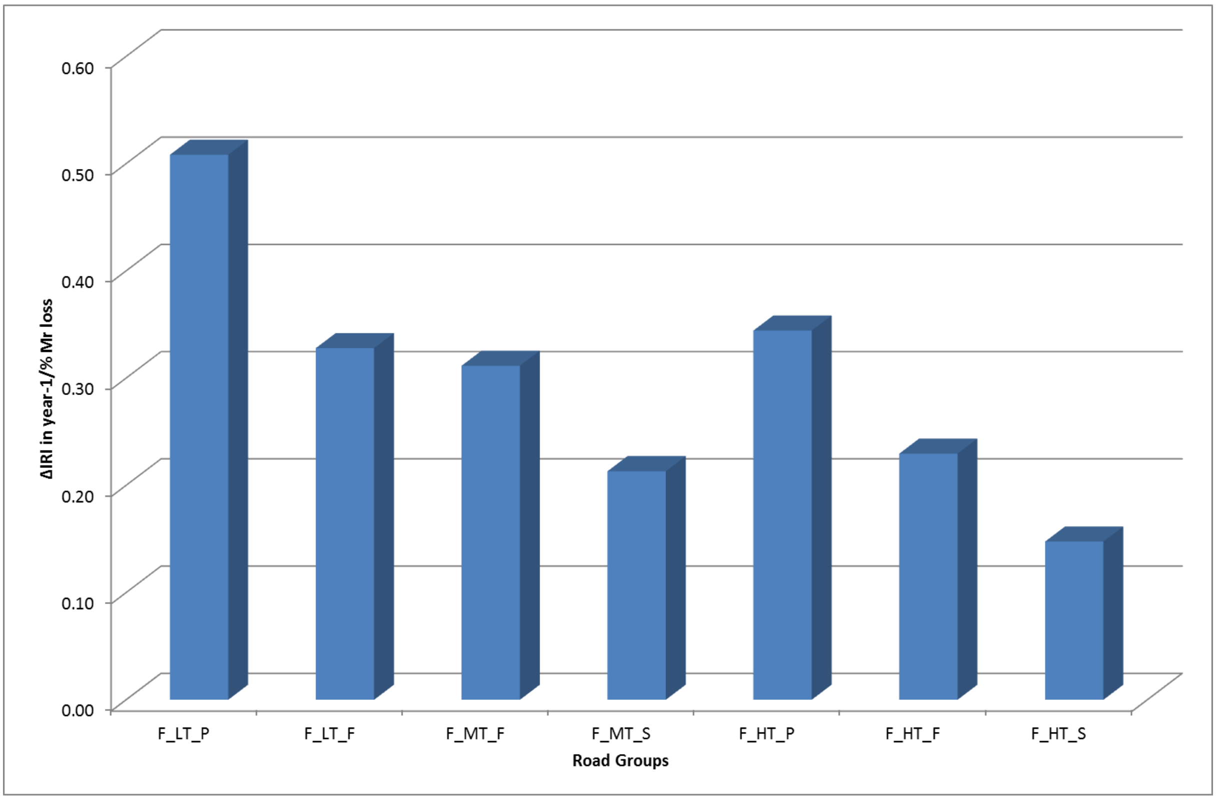


Figure 6.5: Results on ΔIRI/MrL for the flexible pavement road groups

6.4 Proposed Flood-resilient Pavements

Pavement performances after a flood is modelled which indicates that a rigid and strong pavement with a high loading and a higher standard performs the best. The ΔIRI/Pr values of all the road groups have been assessed, which shows the same findings obtained from the pavement responses with flooding. For example, the F_LT_S, R_HT_S and R_MT_F road groups had gradient values of 0.10, 0.14 and 0.18, respectively. It shows that a rigid and strong road performs the best. A strong and low loading road performed well in this case which is reasonable. Detailed results are shown in *Figure 6.7*. It was not possible to extract results for all the road groups because of the absence of consistent data for some road groups in RD modelling. The Mr loss analysis was only done for some flexible pavement road groups.

The study uses a new ΔIRI/MrL to obtain the flood-resilient flexible pavements which are shown in *Figures 6.5* and *6.6*. However, this analysis only covered flexible pavement road groups, which reveals that a strong and high loading with higher standard pavement performs the best with flooding. For example, the F_HT_S, F_MT_S and F_HT_F road groups had gradient values of 0.15, 0.22 and 0.23, respectively.

Both the gradient values, that is, ΔIRI/Pr and ΔIRI/MrL, and the comparison of results on the performances of different pavements types with flooding provided consistent outcomes in obtaining the flood-resilient pavements; which is mainly a rigid and strong pavement having high loading and higher standard. A road authority may try to i) convert a flood damaged road into a rigid one, and ii) increase structural strength through thick overlay and/or granular and subgrade layers' stabilisation.

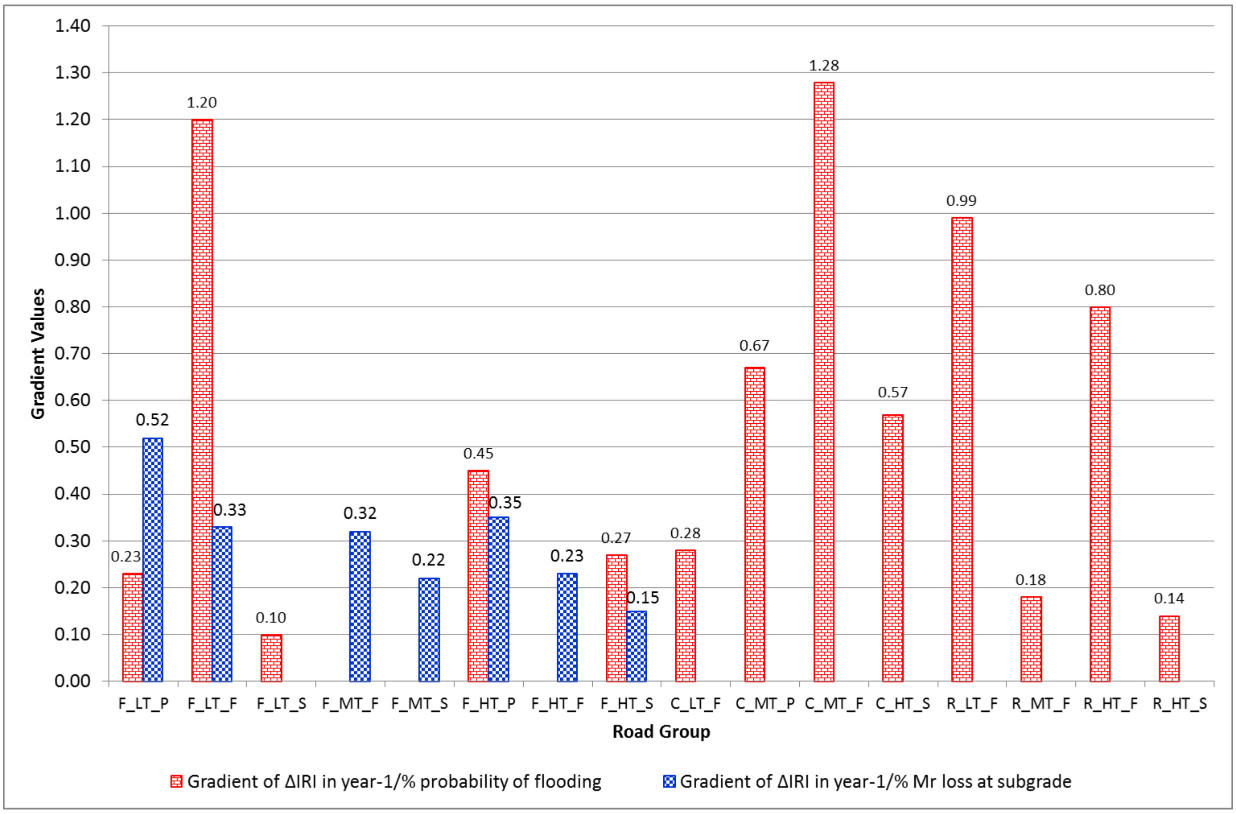


Figure 6.6: Comparison on ΔIRI/Pr and ΔIRI/MrL to obtain pavements flood resilience

The valid RD models results among four road groups are used to show the importance of converting a road in to a flood resilient one. As an example, a Flexible pavement with High Traffic loading and Poor strength (F_HT_P) road has been used here. As it is a poor road, it may be converted to a flood resilient one by enhancing its strength. As a result, a F_HT_P road may be converted to a Flexible pavement with High Traffic loading and Strong strength (F_HT_S) road with asphalt concrete, a Composite pavement with High Traffic loading and Strong strength (C_HT_S) road with layers stabilisation, and a Rigid pavement with High

Traffic loading and Strong strength (R_HT_S) road by changing to a rigid one. Therefore, RD models of these road groups are compared and interpreted logically to reveal benefits of converting roads in to flood resilient ones. Pavements performances after a hypothetically chosen flood with 50% probability have been used. The analysis assumed a flood in year 1, and also very good road condition before the flood. The Appendix 4C shows that the C_HT_S did not have an IRI-based RD model developed. Therefore, the F_HT_P, F_HT_S and R_HT_S results are shown in *Figure 6.7*.

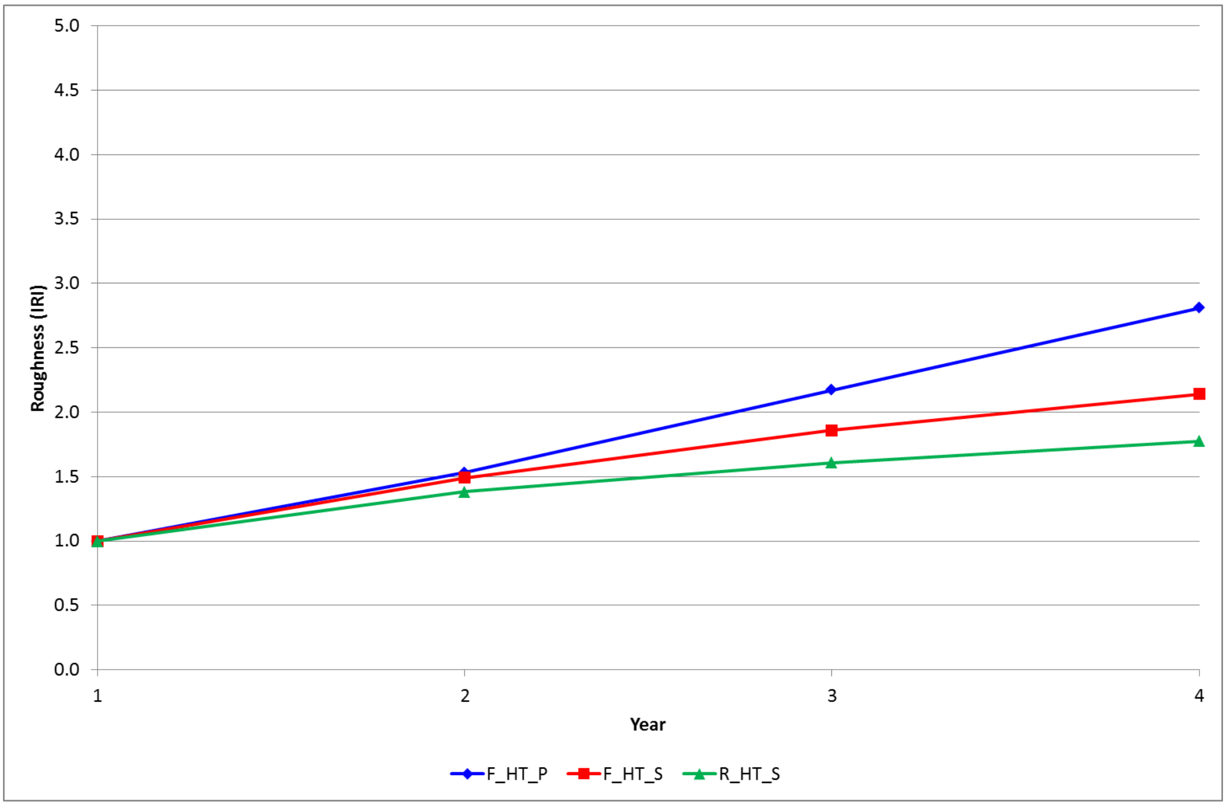


Figure 6.7: performances of non-flood and flood-resilient pavements

The results show that if a F_HT_P road is converted to a flood resilient one, then roughness reduction between year 1 and 2 would be 0.04 to 0.15 IRI. Higher roughness reductions are observed between year 2 and 3 with 0.31 to 0.56 IRI and between year 3 and 4 with 0.67 to 1.03 IRI. In all cases, the R_HT_S provides a higher IRI reduction or better performance than the F_HT_S; hence a rigid flood-resilient pavement shows better performance than a flexible one. The other analysis results of this research reveal that a composite flood-resilient road would perform similar to a flexible one. The converted flood resilient road needs a thinner OV in any year after the flood, instead of a thicker overlay or major rehabilitation that is required for the unconverted F_HT_P road. Therefore, it is clear that a pavement performs better in life-cycle if it becomes flood resilient, and agency costs are also minimised.

6.5 Summary

This chapter highlights how the pavement performances with flooding can be quantified. Two new indicators of ΔIRI/Pr and ΔIRI/MrL were used for this purpose, which were derived from the RD modelling results and assessing pavement performances due to Mr loss. Moreover, the RD models were reviewed to get different types of pavements' performances with flooding. All these assisted obtaining the flood-resilient pavements, i.e., a rigid and strong pavement having high loads and higher standard.

CHAPTER SEVEN: OPTIMUM MAINTENANCE STANDARDS AND STRATEGIES FOR THE TMR-QLD[4]

7.1 Introduction

Developing an optimum maintenance strategy has been discussed in Section 3.4, and the approach is given in *Figure 3.7*. Detailed results of deriving the optimum maintenance standards and strategies at normal conditions for the TMR-QLD are discussed below. *Figure 3.1* shows the components highlighted in this chapter.

Generally, routine and periodic maintenance treatments (includes preventive maintenance and rehabilitation) are utilised to manage roads at set maintenance standards and strategies at the network level. It means that a road network needs to be maintained when it is good to fair, which provides the best economic return on investment. Moreover, a reconstruction is not included in a strategy as it indicates allowance of complete failure for a road without intervening with routine and periodic maintenance; which is not desirable. Therefore, the previously mentioned six treatments as practiced by the TMR-QLD are used for each standard.

The current analysis highlights the results of flexible and composite pavements. It is worth noting that both the flexible and composite representative pavements cover more than 99% of the network. Details of the representative 27 road groups are shown in *Appendix 4B*. About 85 sections were used to derive the nine rigid pavement road groups. Although they were nominal, their pavement performance trends provided with and without flooding TPMs which helped getting RD models for the rigid pavements. These results are adequate for flood resilience and after-flood pavement performances estimation. However, because of their 1% total length and about 84 km of representative road groups' length, the concrete pavements were not used for obtaining the optimum, pre- and post-flood strategies; as these results had little impact on budget.

The standard-treatment alternatives used at a set standard of 3.0 IRI for a road group can be seen below:

4 *These findings provided a Journal Paper: http://www.tandfonline.com/doi/full/10.1080/14488353.2017.1362823*

- RM @ 3.0 IRI
- Seal Coat 15 mm @ 3.0 IRI
- Slurry Seal 10 mm @ 3.0 IRI
- OV 30 mm @ 3.0 IRI
- OV 45 mm @ 3.0 IRI
- OV 75 mm @ 3.0 IRI

Similarly, this same road group will have above six treatment alternatives for each of the set standards of 3.5, 4.0, 4.5, 5.0 and 5.5 IRI, totalling 36 standard-treatment alternatives (six standards and six treatments) to obtain the optimum one. The unit costs used in the analysis are shown below. These are typical average costs based on current practices, and they may be changed due to location, materials, density of distress, pavement condition and planned preventive maintenance. The current research has used these average costs for all the analysis; hence they did not have any impact on the HDM-4 results. In future, more accurate treatment costs may be used to get a better result.

- Pothole patching ($\$/m^2$) = 183.0
- Edge repair ($\$/m^2$) = 45.0
- Crack sealing ($\$/m^2$) = 14.0
- Seal coat ($\$/m^2$) = 30.0
- Slurry seal ($\$/m^2$) = 45.0
- OV 30 mm ($\$/m^2$) = 75.0
- OV 45 mm ($\$/m^2$) = 100.0
- OV 75 mm ($\$/m^2$) = 150.0
- Reconstruction ($\$/m^2$) = 180.0

It is important to note that a RM needs to be done whenever necessary. However, in the analysis this treatment was considered as one of the alternatives. Therefore, different standards were used with RM for the analysis. This is a kind of base option and cannot be chosen by the model as an ideal/optimal solution. The after work effects consider a change in road condition and IRI after a treatment. The HDM-4's WE model have been used in the analyses which have sub-models to estimate the effect of different treatments. These sub-models are discussed in Odoki and Kerali (2000). Generally, seal coat and slurry seal cannot improve roughness, rather they are provided for sealing cracks and protecting the surface, which slow down the deterioration. These treatments are considered in the analysis as they are being practiced. On the other hand, an overlay improves roughness and road condition, and also enhances pavement structural strength.

The HDM-4 road deterioration, work effects and road user effects models were calibrated locally (Rashid, et al. 2013; TMR, 2010). The detailed HDM-4 results are discussed below.

7.2 Results of the HDM-4 Analysis

The current research has shown results for the 18 flexible and composite pavement road groups. It was planned to utilise all the 27 road groups; however, lengths of the rigid pavement representative road groups were found very insignificant. Therefore, it was not possible to include these roads in the HDM-4 analysis. *Figure 7.1* reveals the derived optimum maintenance standards for these road groups. In general, for any pavement type and loading road groups, strong roads have better standards and poor roads have relaxed standards. For example, a Flexible pavement with Low Traffic loading and Strong strength (F_LT_S) had an optimum standard of 4.5 IRI, whereas, a Flexible pavement with Low Traffic loading and Fair strength (F_LT_F) and a Flexible pavement with Low Traffic loading and Poor strength (F_LT_P) had optimum standards of 5.0 and 5.5 IRI, respectively, which are consistent with their road groups. Again, considering varying loading, F_LT_P had an optimum standard of 5.5 IRI, while, Flexible pavement with Medium Traffic loading and Poor strength (F_MT_P) and Flexible pavement with High Traffic loading and Poor strength (F_HT_P) had standards of 5.0 and 4.5 IRI, respectively. This indicates that a high loading road needs a better standard to be managed efficiently. *Figure 7.1* shows that generally, composite roads are to be maintained at higher standards. The detailed economic results and necessary treatments for all the road groups are not given here.

Overall, $17.79bn is needed to maintain the flexible and composite roads in the next 20 years starting from 2014. This means $0.9bn or $0.04 m/km are required in each year. The summarized maintenance strategies for the whole road network are given in *Table 7.1*, which has provided required treatments, trigger years and budget. It is worth mentioning that a RM has to be given whenever necessary, which has been included in the life-cycle analysis for the budget estimation.

Three types of treatments were recommended from the HDM-4 analysis, which was done through a comparison of economic indicators among the 36 standard-treatment alternatives. As most of the roads were proposed to be maintained at high standards, i.e., 3.5-4.5 IRI, 30 mm overlay was found to be the best solution since it is a cheaper strengthening treatment. As expected, a seal coat was suggested for the composite roads with low traffic loading. However, the flexible pavement roads with low traffic loading needed a 30 mm overlay, while, a 45 mm overlay was chosen for the medium loading but poor strength road group at 5.0 IRI. It appeared that the high loading with poor strength road groups needed a 30 mm of thin overlay at 4.5 IRI, but their optimum strategy suggested that OV 30 mm had to be given twice in 2018 and 2032 in its life-cycle (see *Table 7.1*).

Figure 7.1 reveals a relationship between optimum standards and budget in $ m/yr/km. The results show that the flexible pavement groups need more budget than the composite roads. Generally, higher budget is needed to maintain a road at a better standard. For example, $0.047m/yr/km was required to maintain the F_HT_S at 4.0 IRI, whereas, $0.029m/yr/km was necessary to manage the F_LT_S at 4.5 IRI. However, contradictory results were observed for the F_MT_P and F_HT_P road groups where higher budget was needed to maintain these roads at 5.0 and 4.5 IRI (relaxed standards), respectively. These roads are poor and carry a high load, which means a higher budget was required to strengthen and manage them at set standards. Another interesting observation was that at the same standard, high loading roads need higher budget. For example, $0.047m/yr/km was required to maintain the F_HT_S at 4.0 IRI, whereas, $0.036m/yr/km was necessary to manage the F_MT_S at the same standard of 4.0 IRI. Although they have the same standards and similar strength, F_HT_S required a higher budget to carry a higher load.

The results of composite pavement road groups reveal that the roads with high standards and high loadings need higher budget. For example, a C_LT_P needed fewer budget than a C_LT_S, as the latter one has a better standard. Moreover, *Figure 7.1* shows that a C_HT_S needed a higher budget because of the higher loading than C_LT_S. Considering the optimum standards and budget for better road performances; it appears that the flexible pavement road groups are sensitive to strength, whereas, the composite road groups are sensitive to traffic loading.

The results reveal that the average network roughness for the 20 year period is 4.0 IRI if the recommended optimum strategy is followed. *Figure 7.2* shows the average performance of low, medium and high loading flexible pavement road groups. In general the high loading roads are maintained at 3.0 IRI as they are major roads and need much attention. The medium loading roads are maintained at 4.0 to 4.5 IRI, which is justified compared to the high loading road groups. As anticipated, the results show that the low loading roads get relatively less attention. The high and medium loading road groups are to be treated at their set standards around 5th to 8th years; therefore, they perform better than the low loading road groups.

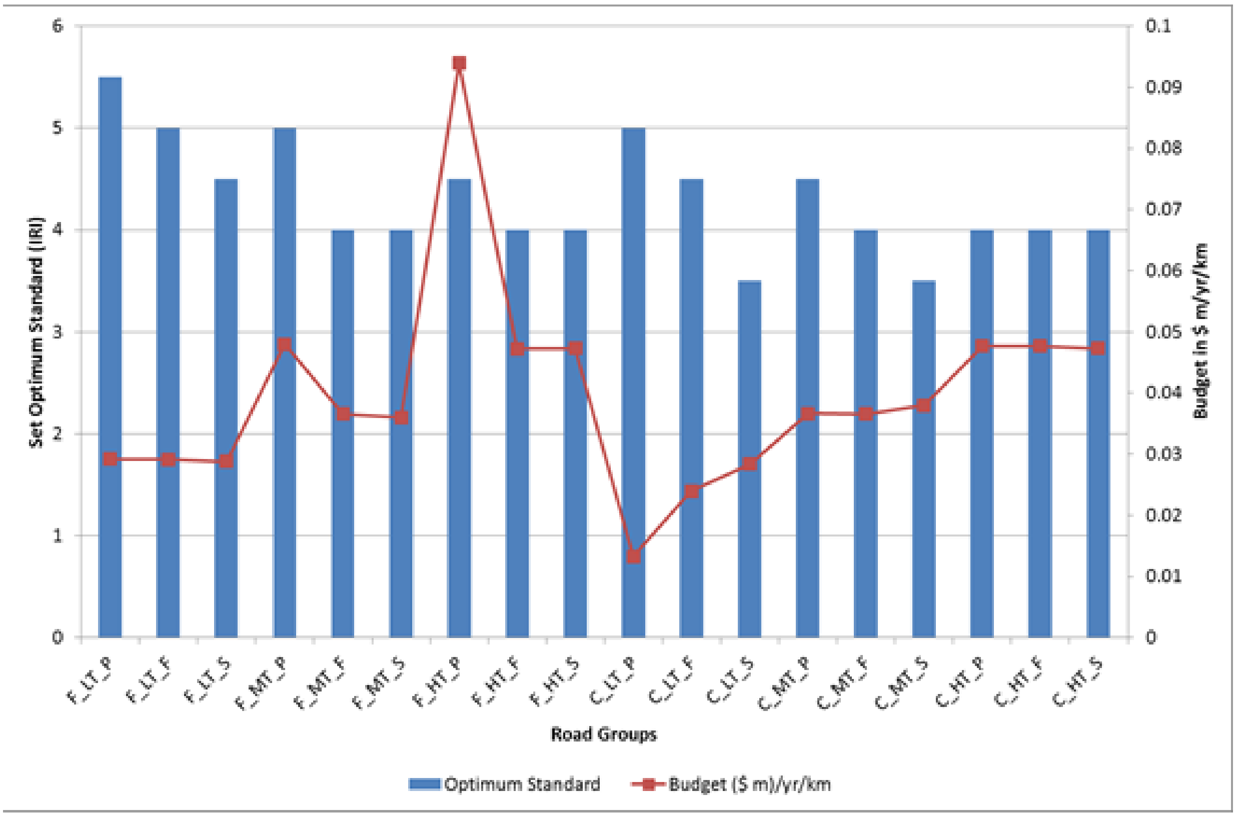


Figure 7.1: Derived optimum maintenance standards and required budget for the flexible and composite roads

Table 7.1: Derived maintenance strategies for the flexible and composite road groups

Treatment Year	Suggested Treatments with Optimum Standards	Road Groups	Description of the Road Groups	Budget Required in Life-Cycle	Average Budget Required (/yr/km)
2018	OV 30 mm @ 4.5 IRI OV 30 mm @ 4.0 IRI	F_HT_P C_HT_P	Flexible, High Traffic loading & Poor Composite, High Traffic loading & Poor	\$507.90 m \$20.02 m	\$0.094 m \$0.048 m
2019	OV 45 mm @ 5.0 IRI OV 30 mm @ 4.0 IRI	F_MT_P C_HT_F	Flexible, Medium Traffic loading & Poor Composite, High Traffic loading & Fair	\$4,178.8 m \$67.91 m	\$0.048 m \$0.048 m
2020	OV 30 mm @ 4.0 IRI OV 30 mm @ 4.5 IRI OV 30 mm @ 4.0 IRI	F_HT_F C_MT_P C_HT_S	Flexible, High Traffic loading & Fair Composite, Medium Traffic loading & Poor Composite, High Traffic loading & Strong	\$913.22 m \$47.10 m \$85.69 m	\$0.047 m \$0.037 m \$0.047 m
2021	OV 30 mm @ 4.0 IRI OV 30 mm @ 4.0 IRI	F_MT_F C_MT_F	Flexible, Medium Traffic loading & Fair Composite, Medium Traffic loading & Fair	\$3,158.8 m \$73.28 m	\$0.037 m \$0.037 m
2022	OV 30 mm @ 4.0 IRI	F_HT_S	Flexible, High Traffic loading & Strong	\$402.36 m	\$0.047 m
2025	OV 30 mm @ 4.0 IRI	F_MT_S	Flexible, Medium Traffic loading & Strong	\$1,827.7 m	\$0.036 m
2027	OV 30 mm @ 5.5 IRI OV 30 mm @ 3.5 IRI	F_LT_P C_LT_S	Flexible, Low Traffic loading & Poor Composite, Low Traffic loading & Strong	\$2,569.7 m \$52.94 m	\$0.029 m \$0.028 m
2030	OV 30 mm @ 5.0 IRI OV 30 mm @ 4.5 IRI	F_LT_F F_LT_S	Flexible, Low Traffic loading & Fair Flexible, Low Traffic loading & Strong	\$2,154.7 m \$883.7 m	\$0.029 m \$0.029 m
2031	OV 30 mm @ 3.5 IRI	C_MT_S	Composite, Medium Traffic loading & Strong	\$309.28 m	\$0.038 m
2032	OV 30 mm @ 4.5 IRI Seal Coat 15 mm @ 4.5 IRI	F_HT_P C_LT_F	Flexible, High Traffic loading & Poor Composite, Low Traffic loading & Fair	\$507.90 m \$14.16 m	\$0.094 m \$0.024 m
2033	Seal Coat 15 mm @ 5.0 IRI Seal Coat 15 mm @ 4.5 IRI	C_LT_P C_LT_F	Composite, Low Traffic loading & Poor Composite, Low Traffic loading & Fair	\$0.80 m \$14.16 m	\$0.013 m \$0.024 m
TOTAL				**\$17,790.1m (\$17.79bn)**	**\$0.04m/yr/km)**

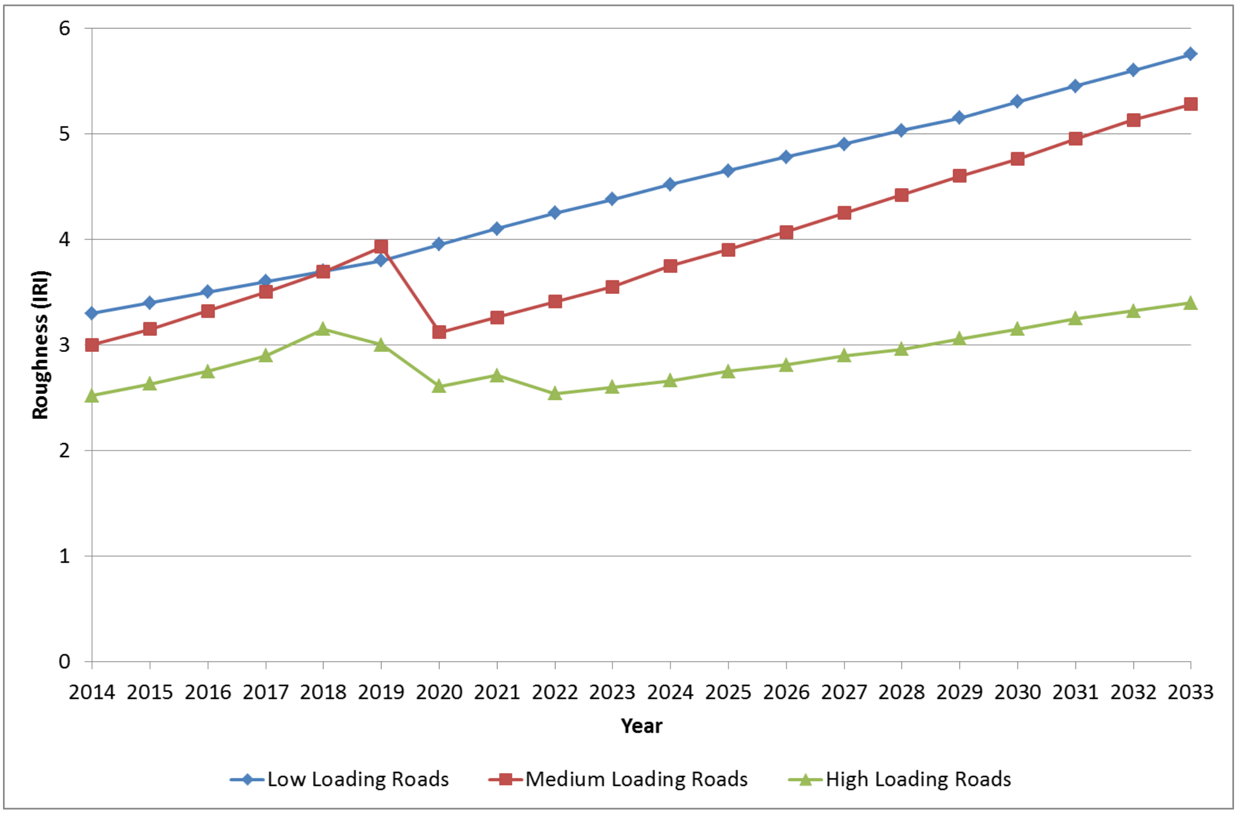


Figure 7.2: Performance of the low, medium and high loading road groups in their life-cycles

7.3 Example of a Road Group

Detailed results of the F_MT_P road group are discussed here as an example, which is one of the common road groups in Queensland. This road group represents about 4,344 km of roads or 18.30% of the whole network. The HDM-4 analysis with the 36 standard-treatment alternatives revealed that this road group had to be maintained at 5.0 IRI with OV 45 mm.

Figure 7.3 shows that among all the standard-treatment alternatives, OV 45 mm at 5.0 IRI produced the highest economic benefit, i.e., NPV/Cost of 1.25. This treatment would give an NPV of $3,853m and an IRR of 16.6%. Therefore, 5.0 IRI was chosen as the optimum standard and RM + OV 45 mm as the suggested maintenance strategy in its life-cycle. The analysis revealed that the F_MT_P needs $4,178.8m budget in the next 20 years, which was $208.9m per year or $0.048m/yr/km budget.

The pavement life-cycle performance of F_MT_P with an optimum standard (5.0 IRI) and strategy (RM + OV 45 mm) can be seen in *Figure 7.4*. It shows that an OV 45 mm is needed in 2019 to keep this road group at the set standard. Further budget optimisation analysis has been done for this road group with this strategy. An OV 30 mm was possible if 75% budget optimisation was considered, as it was cheaper than OV 45 mm. Although this can ensure managing the road group at 5.0 or 5.5 IRI, it provided poor life-cycle performances compared to the strategy with OV 45 mm. Thus, if 100% budget ($4,178.8m) was allocated, this road group could be maintained at 5.0 IRI as recommended. However, if the budget was reduced to 75%, then this road group could not be maintained as planned (see *Figure 7.4*). It indicates that either lower treatment has to be given with an allowance of more deterioration, or no treatment is possible at all if the set strategy is considered. It is worth noting that a NPV is related to life-cycle road user costs and benefits, i.e., VOC, travel time costs, accident costs and respective benefits. If the NPV is negative, then NPV/Cost is negative too.

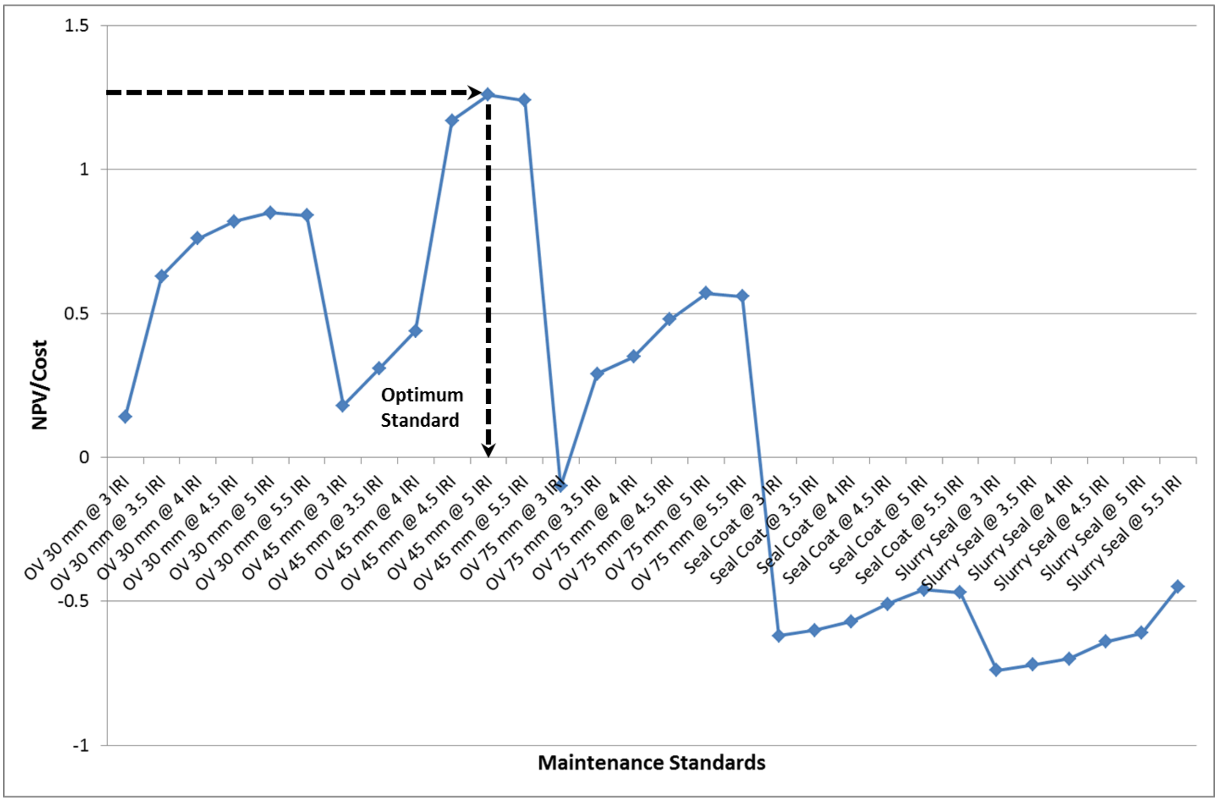


Figure 7.3: Derived optimum maintenance standard for the F_MT_P road group

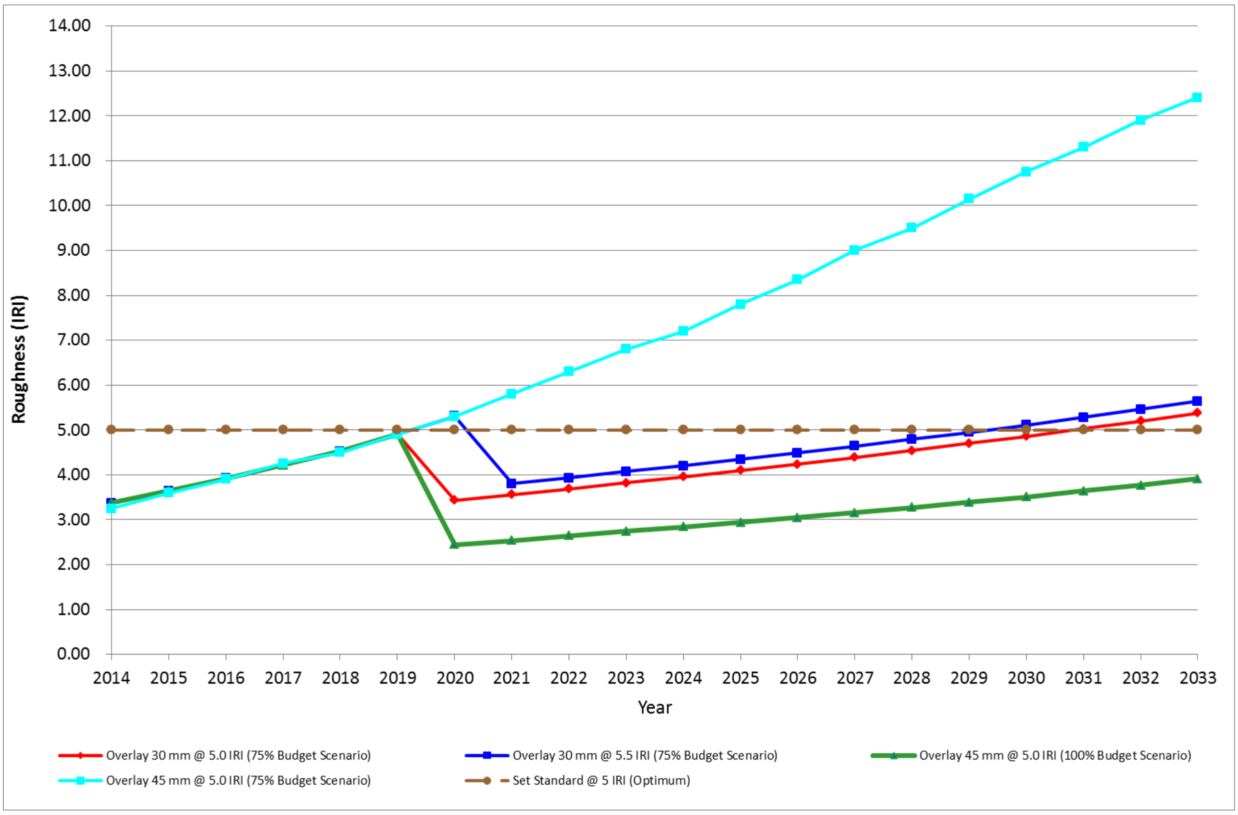


Figure 7.4: Impact of budget on pavement performances for the F_MT_P road group

7.4 Comparison of Results with a Previous Study

The CRC CI study generated maintenance strategies for the TMR-QLD using a sample of 4,500 km of roads (CRC CI, 2006). However, the study did not derive optimum standards at set IRI, rather standards were developed statistically using the HDM-4 outputs with 50% and 83% confidence. In addition, the study used compound strategies along with a reconstruction. Therefore, the CRC CI result was not an optimum strategy. However, they provided useful findings for the same road network which were compared with the seven relevant road groups' results used in the current study (see *Table 7.2*).

The road grouping in the CRC CI study was based on pavement type, surface type, roughness-based road condition, traffic volume and soil types (CRC CI, 2006). Therefore, it was not easy to match that study's road groups with the current one. A simple assumption was used to convert traffic volume into traffic loading; where <1,500 AADT was considered low traffic loading, 1,500 – 10,000 AADT medium traffic loading and > 10,000 AADT high traffic loading. Using the AustRoads pavement structural design guideline (AustRoads, 2010) with about 10% heavy vehicle and 1% growth rate as common in the TMR-QLD's roads, it appears that the design loading would be 1.51 MESAL for 1,500 AADT. It gives a design loading of 10.00 MESAL if 10,000 AADT is used. As a result, the proposed AADT ranges were comparable with the loading groups used in this research. A Wet Non-reactive (WNR) soil type was considered as it is very common. Moreover, the roughness-based road condition value was used as pavement strength. As a result, 13 road groups from the previous study could be matched with the seven road groups of the current study.

Table 7.2 reveals the optimum standards, suggested treatments and budget in $ m/yr/km for the matched road groups. The current study has suggested higher standards to maintain roads at a lower IRI, which is promising. It indicates that the road network will be at good to fair condition using suggested OV 30 mm. The latter study recommended relaxed standards, which called for a thicker overlay of 50 mm to manage the roads. The recommended total budgets of both the studies for these road groups were highly comparable.

It is worth noting that both analyses used similar routine and periodic maintenance treatments apart from a reconstruction considered in the earlier study. However, this treatment was omitted in the current analysis as allowing a road's complete failure and managing it with a reconstruction is a suboptimal treatment. The recent study uses ‘minimise agency cost at target IRI’ as the optimisation objective.

The current study has provided optimum economic standards and strategies (recommended treatments) for these road groups. However, the CRC CI study provided maintenance strategies with a combination of treatments, including reconstruction (CRC CI, 2006), which were not optimum and allowed roads to deteriorate completely. As a result, pavement performances were observed to be poor in the earlier study. As an example, the pavement performance of the F_HT_S road group is shown in *Figure 7.5*. It appeared that it could be maintained at 4.0 IRI with the current strategy (RM + Overlay 30 mm), and it provides positive economic benefits. On the other hand, it would need several combinations of treatments to maintain the road with the CRC CI strategy. Even then, the road could be maintained at 5.0 IRI but the agency costs would be very high as it would need four treatments including reconstruction. Although no economic indicator results were given in the earlier study, it can be anticipated from the higher agency costs and poor pavement performances that the previous strategy was not optimal.

Table 7.2: Comparison of results between the current and previous study

Current Study				CRC CI Study (CRC CI, 2006)			
Road Groups	**Opt. Stand.**	**Suggested Treatments**	**Budget ($ m/yr/km)**	**Road Groups**	**Opt. Stand.**	**Suggested Treatments**	**Budget ($ m/yr/km)**
F_MT_F	4.0 IRI	Overlay 30 mm in 7th yr	0.0366	WNR-Fair-Bt-Flx-(1.5-3 K) WNR-Fair-Bt-Flx-(5-10 K)	5.5 IRI	Resealing in 5th yr (100% area) Granular overlay in 16th yr (100% area)	0.0372
F_MT_S	4.0 IRI	Overlay 30 mm in 11th yr	0.0360	WNR-Good-Bt-Flx-(1.5-3 K) WNR-Good-Bt-Flx-(3-5 K) WNR-Good-Bt-Flx-(5-10 K)	5.0 IRI	Resealing in 3rd yr (50% area) Resealing in 5th yr (70% area) Granular overlay in 13th yr (60% area)	0.0232
F_HT_F	4.0 IRI	Overlay 30 mm in 6th yr	0.0472	WNR-Fair-Bt-Flx-(10-25 K)	5.0 IRI	Overlay 50 mm in 0 yr (42% area) Crack sealing in 3rd yr (58% area) Overlay 50 mm in 4th yr (42% area) Reconstruction in 9th yr (58% area)	0.0488
F_HT_S	4.0 IRI	Overlay 30 mm in 8th yr	0.0473	WNR-Good-Bt-Flx-(10-25 K)	5.0 IRI	Crack sealing in 3rd yr (97% area) Overlay 50 mm in 3rd yr (3% area) Overlay 50 mm in 6th yr (3% area) Reconstruction in 9th yr (97% area)	0.0440
C_MT_F	4.0 IRI	Overlay 30 mm in 7th yr	0.0366	WNR-Fair-Bt-SR-(1.5-3 K) WNR-Fair-Bt-SR-(3-5 K) WNR-Fair-Bt-SR-(5-10 K)	5.5 IRI	Granular overlay in 8th yr (100% area) Granular overlay in 17th yr (100% area)	0.0328
C_MT_S	3.5 IRI	Overlay 30 mm in 17th yr	0.0380	WNR-Good-Bt-SR-(1.5-3 K) WNR-Good-Bt-SR-(5-10 K)	5.5 IRI	Resealing in 6th yr (65% area) Granular overlay in 11th yr (71% area)	0.0292
C_HT_F	4.0 IRI	Overlay 30 mm in 5th yr	0.0478	WNR-Fair-Bt-SR-(10-25 K)	5.0 IRI	Overlay 50 mm in 0 yr (61% area) Crack sealing in 3rd yr (39% area) Overlay 50 mm in 4th yr (61% area) Reconstruction in 9th yr (39% area)	0.0536

- As an example, WNR-Fair-Bt-Flx-(1.5-3 K) means Bituminous and Flexible road with Fair condition, Wet Non-reactive soil and 1,500 to 3,000 AADT; and WNR-Fair-Bt-SR-(5-10 K) means Composite road with Fair condition, Wet Non-reactive soil and 5,000 to 10,000 AADT.

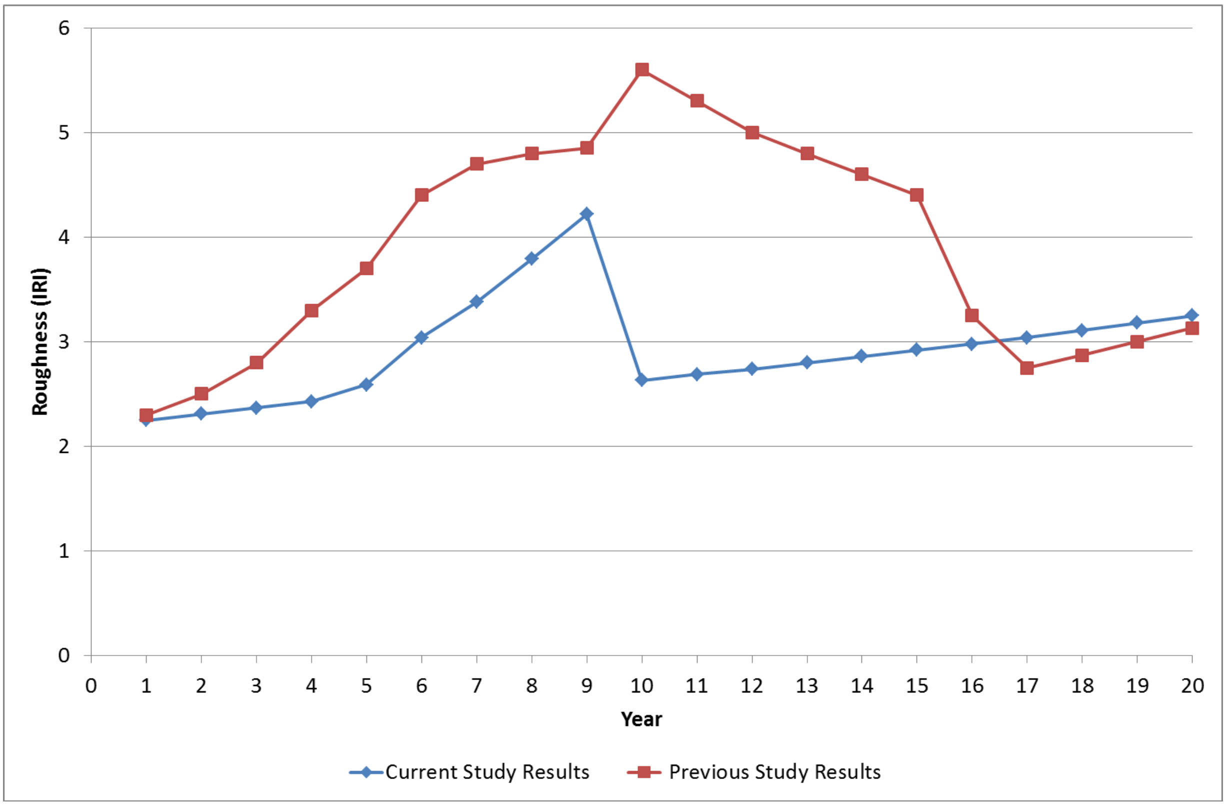


Figure 7.5: Pavement performances at the suggested optimum strategies of the current and previous studies for the F_HT_S

7.5 Summary

Any road authority requires managing its road network at the set standards and strategies. The current research has developed optimum maintenance standards and strategies at without flood condition for the flexible and composite pavements of TMR-QLD, which are shown in this chapter. The analysis used 36 standard-treatment alternatives to get the optimum standard and strategy for a road group. The derived strategy, pavement performances and economic results were compared with a relevant previous study, and found suitable. Finally, a pavement performance at a set standard has been shown.

CHAPTER EIGHT: POST-FLOOD ROAD MAINTENANCE STRATEGY FOR THE TMR-QLD[5]

The TMR-QLD assessed its damaged road network after the 2010/2011 flood, and identified about 6,709 km for reconstruction at $4.2bn (TMR, 2012a and 2012b). The Transport Network Reconstruction Program (TNRP) progress report of June 2012 showed the post-flood rehabilitation program, funding and progress. This program was based only on the after flood road condition data. About 39% reconstruction/rehabilitation were completed after one and half years of the flood, and about $3.4bn was spent or 70% of work was done after two and half years (TMR, 2012b). The project has been completed in 2014, and took over 3 years. A detailed TNRP map is given in *Appendix 8A*.

Based on the post flood rehabilitation implementation experiences, the current research assumed a flood in year 1 and rehabilitation in year 2, 3 or 4 to address the reality. The approach considered to get a post-flood road maintenance strategy was shown in *Figure 3.8*. The items discussed here are shown in *Figure 3.1*.

The TNRP program has some limitations: i) it considered the road condition data of 2011/12; ii) did not have a tool to predict roads deterioration appropriately up to 2 to 3 years after a flood; and iii) could not use an optimum strategy, as no economic analysis was used. These limitations are valid for any road authority because of not having appropriate RD models and subsequent post-flood maintenance strategy.

The current analysis has used the new roughness and rutting based RD models with flooding to obtain pavement performances after a flood (developed in Chapter four), and then used the HDM-4 model for a post-flood strategy. It is worth highlighting that road conditions (especially roughness and rutting) after a flood were predicted using the new RD models, not by the HDM-4 deterioration models. The post-flood strategies were either constrained or unconstrained budget solutions. It assumed a flood in year 1 and then necessary post-flood rehabilitation may start from year 2. The detailed approach has been shown in *Figure 3.8.* The current chapter provides all the major findings.

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[5] *The findings provided in this chapter has resulted a Journal Paper: http://dx.doi.org/10.1080/10298436.2015.1121781*

8.1 Recommended Solutions with the Constrained and Unconstrained Budgets

The required life-cycle budget obtained from the post-flood strategy using three optimisation objectives are shown in *Table 8.1*. As mentioned in Chapter Three, the HDM-4 uses three objectives for optimisation, i.e., i) maximise NPV; ii) minimise ΔIRI; and iii) minimise agency cost at target IRI. The analysis provided results for the flexible and composite pavements, totalling 18 road groups, which covered about 99% of the network. Pavement performances was also analysed with a 67% budget constrained scenario, the result of which is given in *Table 8.1*.

Different optimisation objectives have different aims to achieve; as a result, they provide different recommendations for a road group. Considering the key factors, i.e., agency costs, budget, economic benefits, life-cycle performances and suggested treatments, the 'minimise agency cost at target IRI' optimisation objective results may be chosen as the final option. A road authority may assess all the flood damaged roads before implementing any of the strategies, basically solutions with the constrained and unconstrained budgets. Summarised comments on the derived post-flood strategies are also shown in *Table 8.1*.

As 'maximise NPV' only considers positive NPV/Cost, it suggested relaxed treatments for some road groups at the later years, to delay for some and not to consider low traffic loading road groups. Therefore, after-flood road rehabilitation was not properly reflected. This strategy does not use agency cost and pavement performances as constraints in optimisation, and would keep the network at 6.0 IRI. Although this strategy provided economic benefits of $5.35bn, but they were not useful as it did not address all the flood damaged roads with right treatments. This strategy results is given in *Appendix 8B*.

The optimisation objective of 'maximise ΔIRI' maximises roughness reduction for all roads; hence, it requires the highest budget in the life-cycle. This objective considers an unconstrained budget and suggests for the possible strongest treatments. The above post-flood strategy indicates that each road group requires a very high and strong treatment of OV 200 mm at 3.0 IRI either in year 2, 3 or 4. It is in fact a strengthening overlay that increases pavement strength, reduces roughness and sets a road at good condition. Although OV 200 mm with stabilisation is the strongest treatment, the HDM-4 results suggested OV 200 mm only, as it increases pavement structural strength to tackle tensile and compressive strain. In addition, it was cheaper than an OV 200 mm with stabilisation. It is worth noting that both thinner and thicker overlays of 30 mm, 45 mm, 75 mm, 100 mm, 150 mm and 200 mm were used in the analysis. The total required budget of $49.7bn in the life-cycle remained unchanged if treatments were given in any of these years. This post-flood strategy is shown in *Appendix 8C*.

Similarly, 'minimise agency cost at a target IRI' provided the strongest possible treatments for the composite roads and suggested relaxed treatments in the later years to the flexible pavement road groups. Another separate analysis obtaining the flood resilient pavements showed that the composite pavements did not behave better than the flexible ones. As a result, this post-flood strategy suggested composite pavements to be rehabilitated mostly in year 2 with OV 200 mm at 3.0 IRI. Flexible pavements required OV 30 mm at 4.0 IRI if road conditions were acceptable; otherwise, the high traffic loaded flexible pavement groups needed reconstruction in between 8 and 12 years. It is worth noting that the analysis used a reconstruction as a base case solution, and this treatment was chosen in the later years when a post-flood rehabilitation was not suggested in the HDM-4 optimisation. This strategy requires $26.1bn in the life-cycle. Although this post-flood strategy provided a loss of $7.05bn, it considers both the agency cost and performance target as constraints during optimisation. The road network would be at 5.2 IRI if this strategy was considered. The strategy manages roads at set targets with constrained budget, and hence it is an optimal solution. A road authority may consider this option after thorough investigation of its flexible roads. *Appendix 8D* shows the results derived using this optimisation objective.

The current analysis has also derived another set of post-flood strategy with budget optimisation. The 67% budget optimisation or about $17.4bn budget was considered for a budget optimisation analysis to get the post-flood budget constrained strategy results. This budget was used as it was close to the normal funding of the TMR-QLD. This strategy would keep the network at 6.4 IRI, the details of which are given in *Appendix 8E*.

The developed optimum maintenance strategy with 'minimise agency cost at target IRI' at no flood condition for the TMR-QLD reveals that about $17.8bn is needed in the next 20 years; whereas $26.1bn is needed if one flood was assumed in the life-cycle. The current yearly budget of TMR-QLD was found consistent to the proposed optimum budget of about $0.9bn per year. The TNRP report shows that about $4.2bn was needed to rehabilitate 6,709 km of roads for the 2010/11 flooding; whereas, the current HDM-4 post-flood strategy provides that an additional \$8.3bn was needed in life-cycle for rehabilitating 23,660 km of representative flexible and composite pavements if a flood event was included. Therefore, the current result was comparable with the post-flood rehabilitation program which was undertaken by the TMR-QLD.

Additional maintenance costs at no flood conditions would be required until a flood comes in any year instead of year 1. Again, if two or more flood events are found, then a separate post-flood rehabilitation is required with extra costs. The current approach to get a post-flood strategy is valid for each flooding event.

The above results provided two solutions considering constrained and unconstrained budget. The unconstrained solution suggested the possible strongest treatments and could ensure the best pavement performances in the life-cycle. On the other hand, the constrained budget one kept the road network at a set target by minimising agency cost. Considering agency cost, funding and performances, it was a realistic solution. Moreover, its costs' matched with the TNRP program of TMR-QLD.

As an example, results of the Flexible pavement with Medium Traffic loading and Poor strength (F_MT_P) road group covering a huge portion of the network (4,344 km) are shown in *Table 8.2*. The road group had set standard of 5.0 IRI and it needed OV 45 mm in year 8 at 5.0 IRI with a budget of \$4.18bn when no flood was observed. The three optimisation objectives provided different solutions if a flood was assumed in year 1. The unconstrained budget using maximise ΔIRI suggested OV 200 mm at 3.0 IRI in year 2 at \$9.70bn. The 'maximise NPV' optimisation objective allowed the road to deteriorate after a flood without doing any rehabilitation, as a result it needed a reconstruction in year 10. Again, 'minimise agency cost at target IRI' suggested a thinner overlay of 30 mm at 4.0 IRI in year 6 at \$3.10bn. The 67% budget optimisation strategy recommended OV 45 mm at 5.5 IRI in year 3 at \$4.16bn. As mentioned above, this budget was close to the current funding; therefore, almost the same cost was needed for no-flood and the 67% budget constrained strategy to manage this road group. Similar to the F_MT_P, another road group of Flexible pavement with High Traffic loading and Strong strength (F_HT_S) had the unconstrained budget solution with OV 200 mm at 3.0 IRI in year 4 at \$1.25bn (see *Figures 8.1 to 8.3*). The 'maximise NPV' suggested for OV 30 mm at 4.0 IRI, and a reconstruction was required in year 10 if the ‘minimise agency cost at target IRI’ was used. Moreover, the 67% constrained budget strategy suggested OV 30 mm at 3.5 IRI in year 7 with $0.4bn.

Table 8.1: Summarised remarks on the derived post-flood maintenance strategies

Optimisation Objective	**Agency Cost**	**Budget Requirement**	**Economic Benefits**	**Performance Target**	**Suggested Treatments**	**Remarks**
Maximise NPV (to get maximum economic benefits)	Not minimised	$16.9bn (constrained)	+$5.35bn +ve	6.0 IRI (Bad)	Relaxed and at late years Delayed Not all the road groups were selected Might not be appropriate for the flood damaged roads	Performance was not addressed. Appropriate treatments were not chosen. Not useful for the flood-damaged roads.
Minimise ∆IRI (to reduce roughness change)	Not minimised	$49.7bn (unconstrained and unrealistic)	-$27.7bn -ve (very high)	1.5 to 2.0 IRI (Excellent to Good)	Possible strongest treatments Best solution	Budget is always a constrained. Might not be a practical one.
Minimise agency cost at target IRI	Minimised	$26.1bn (constrained)	-$7.1bn -ve	5.2 IRI (Poor)	Relaxed and at late years Delayed Might not be appropriate for all the road groups	Considers agency cost, budget and performance in optimisation. Practical. Might be used after a detailed investigation.
Budget constrained (it is not an optimisation objective, rather 67% budget was assumed available)	Minimised (indirectly)	$17.4bn (constrained)	+$8.98bn +ve	6.4 IRI (Bad)	Relaxed and at late years Delayed Might not be appropriate for all the road groups	Performance was not addressed. Budget optimum solution. Might be used after a detailed investigation.

Table 8.2: Post-flood strategy derived for the F_MT_P and F_HT_S

Road Group	Strategy with Recommended Treatment & Budget		
F_MT_P	Normal Condition	Optimum Strategy	OV 45 mm at 5.0 IRI in year 8 at $4.18bn
	Flooding Condition	Unconstrained budget strategy using 'minimise ΔIRI'	OV 200 mm at 3.0 IRI in year 2 at $9.70bn
		Strategy using 'maximise NPV'	Reconstruction in year 10 at $2.42bn
		Constrained budget strategy using 'minimise agency cost at target IRI'	OV 30 mm at 4.0 IRI in year 6 at$3.10bn
		Strategy with 67% constrained budget	OV 45 mm at 5.5 IRI in year 3 at $4.16bn
F_HT_S	Normal Condition	Optimum Strategy	OV 30 mm at 4.0 IRI in year 9 at $0.40bn
	Flooding Condition	Unconstrained budget strategy using 'minimise ΔIRI'	OV 200 mm at 3.0 IRI in year 4 at $1.25bn
		Strategy using 'maximise NPV'	OV 30 mm at 4.0 IRI in year 8 at $0.40bn
		Constrained budget strategy using 'minimise agency cost at target IRI'	Reconstruction in year 10 at $0.31bn
		Strategy with 67% constrained budget	OV 30 mm at 3.5 IRI in year 7 at $0.40bn

8.2 Pavement Performances with the Post-flood Strategy

Pavement life-cycle performances for the 18 road groups with the proposed unconstrained budget solution as a post-flood strategy are shown in *Figure 8.1* where 16 road groups required post-flood rehabilitation in year 2 assuming a flood in year 1. Two road groups, i.e., F_HT_S and Composite pavement with High Traffic loading and Strong strength (C_HT_S), needed treatments in year 3. Deterioration of these road groups in year 2 and up to year 3 for the two road groups were predicted using the new RD models with flooding. It is worth noting that the first year data was extracted from the real database, and then a 100% probability of flooding (or a certain flood) was assumed in year 1. As a result, an abrupt increase in roughness was observed in year 2 and 3. Then, the HDM-4 model provided the post-flood strategy for each road group considering the 78 standard-treatment alternatives. Although this strategy delivered an economic loss of $27.7bn, it addressed all the flood damaged roads and suggested one of the strongest treatments to keep the network at an excellent to good condition.

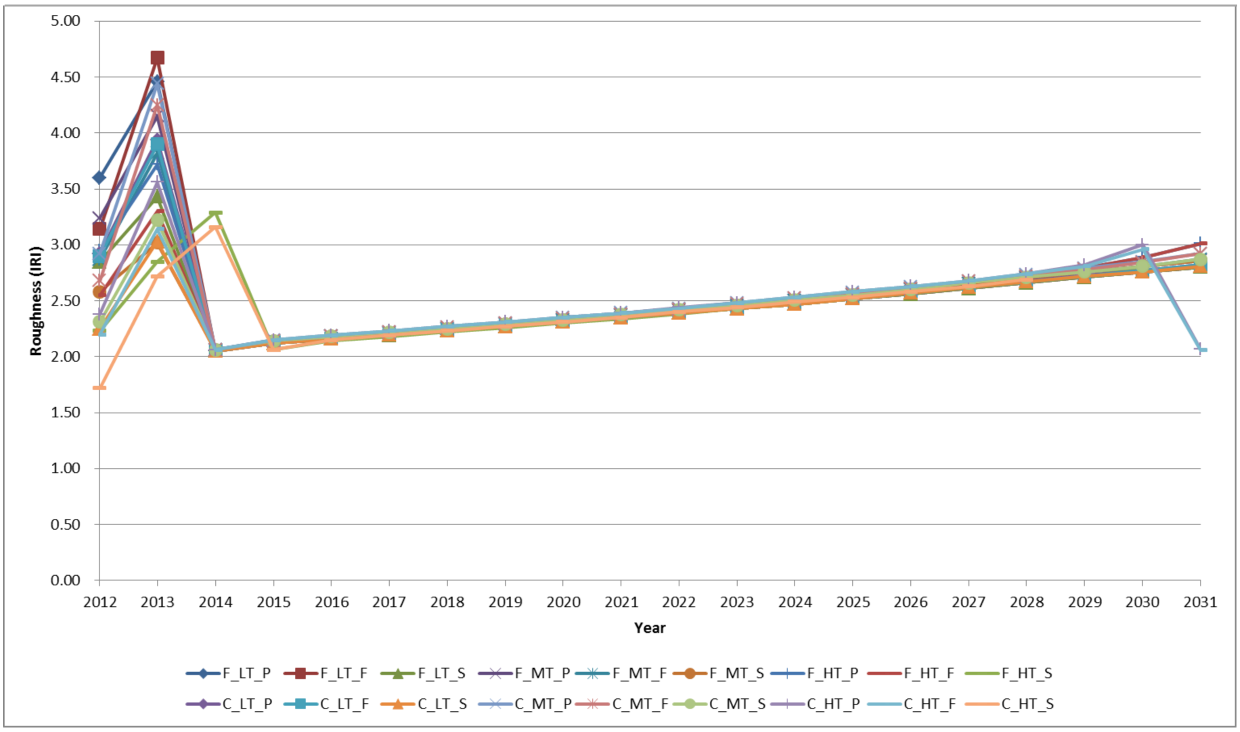

Figure 8.1: Pavement deterioration prediction after a flood for year 2 using the new RD models assuming a flood in year 1 and rehabilitation starts from year 2

Similarly, *Figures 8.2 and 8.3* show pavements' life-cycle performances for these roads assuming a flood in year 1 and rehabilitation in year 3 and 4 respectively if budget is an issue. Pavement deteriorations up to year 3 and 4 respectively were predicted using the developed RD models, as the HDM-4 can only consider a normal deterioration. The current practice also considers a normal deterioration after a flood event; whereas, the new RD models show higher deterioration at different probabilities of flooding up to 2 to 3 years after a flood. Therefore, the new RD models predict real deterioration which helps developing a realistic post-flood strategy.

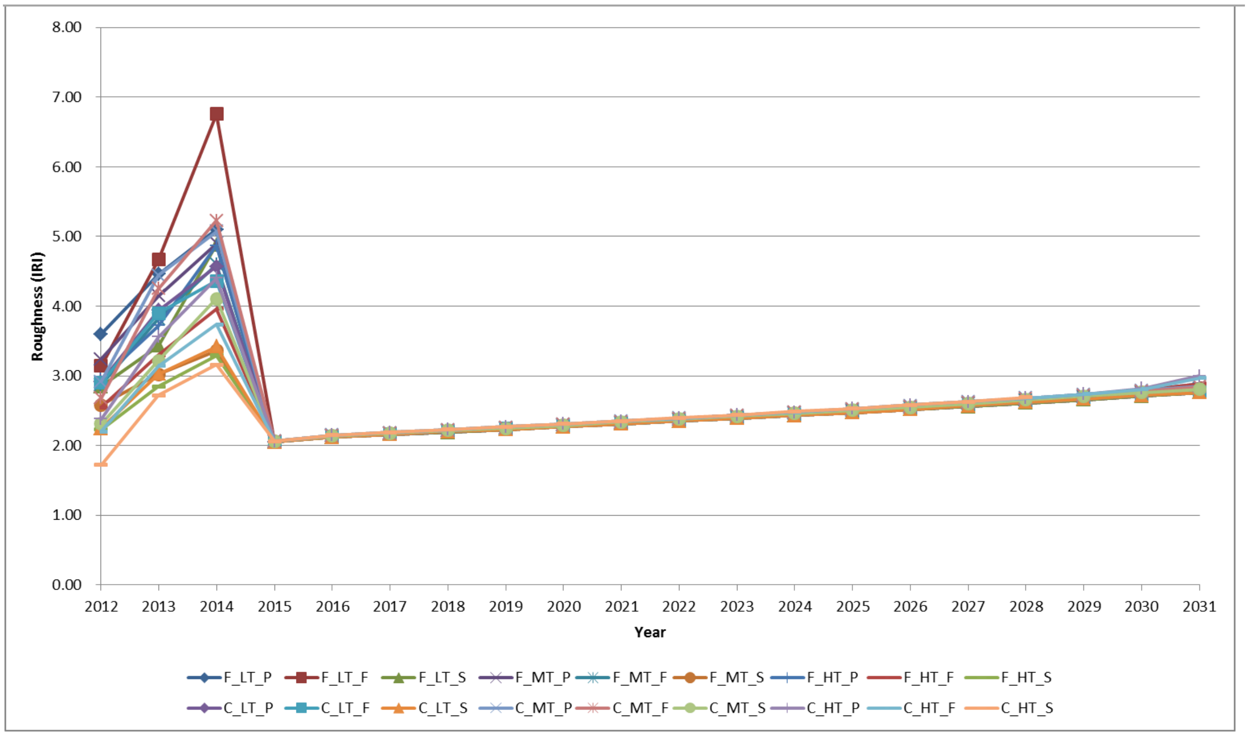

Figure 8.2: Pavement deterioration prediction after a flood for year 2 and 3 using the new RD models assuming a flood in year 1 and rehabilitation starts from year 3

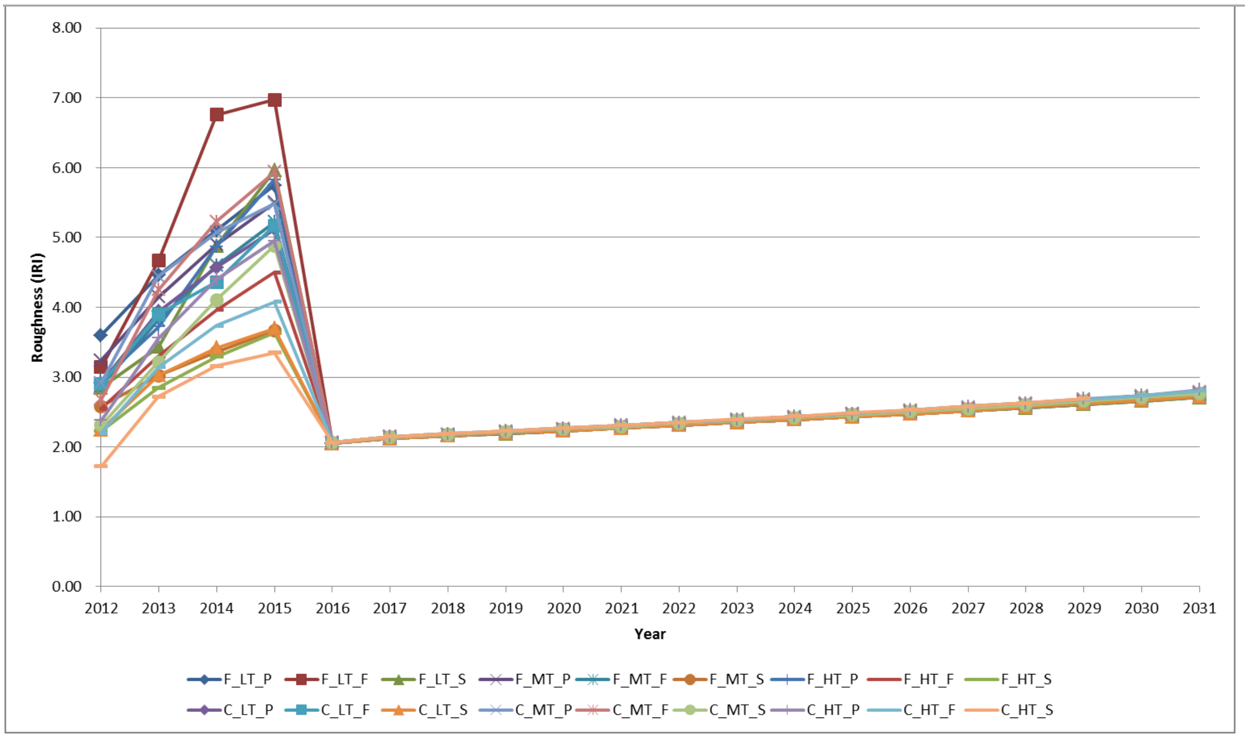

Figure 8.3: Pavement deterioration prediction after a flood for year 2, 3 and 4 using the new RD models assuming a flood in year 1 and rehabilitation starts from year 4

Performance of the whole road network with these post-flood strategies can be seen in *Figure 8.4*. Although, the unconstrained budget strategy using 'maximise ΔIRI' provided the best performances, it needed $49.7bn which is not easy to get. Considering the comparison given in *Table 8.2* and the performances shown in *Figure 8.5*, the constrained budget strategy with 'minimise agency cost at target IRI' would keep the network at a reasonable condition.

Figure 8.5 shows pavement performances for the two sampled road groups, i.e., F_HT_S and F_MT_P, where both the constrained and unconstrained budget solutions were used. Considering budget, performances and agency costs, the constrained budget solution seems practical and near optimum.

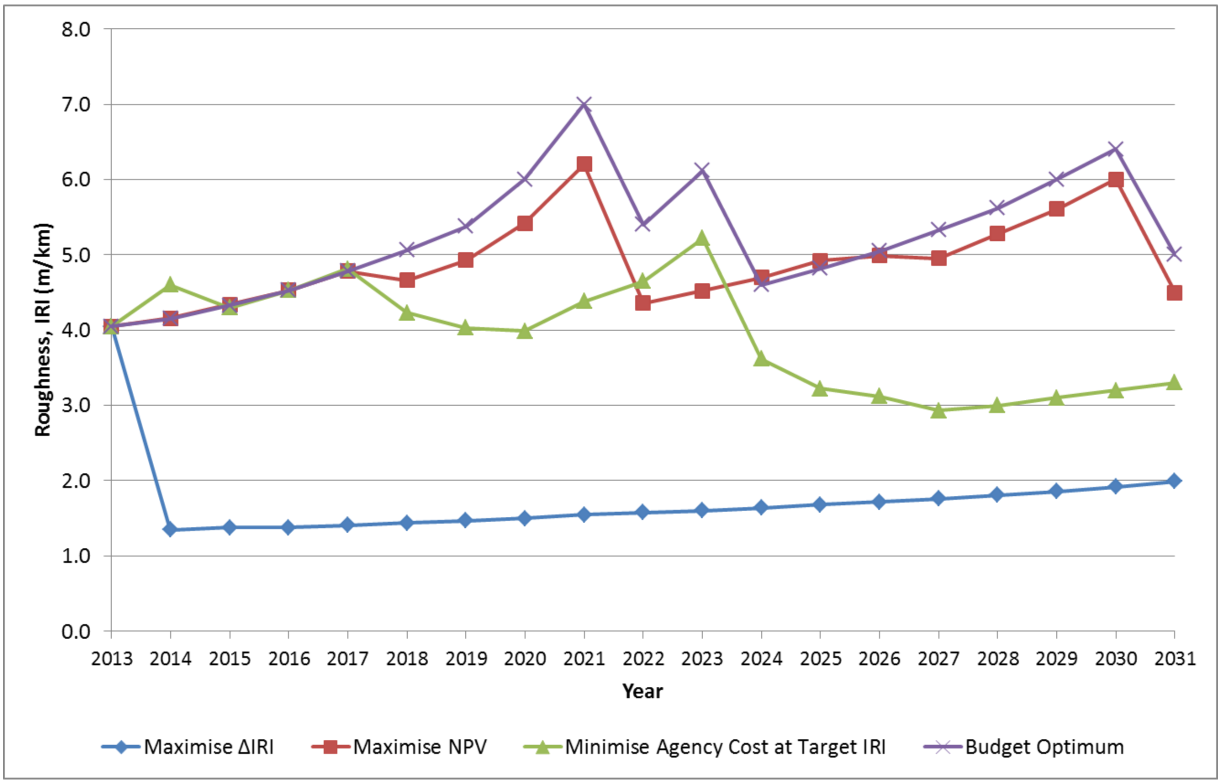

Figure 8.4: Life-cycle performances of the road network with the derived post-flood strategies

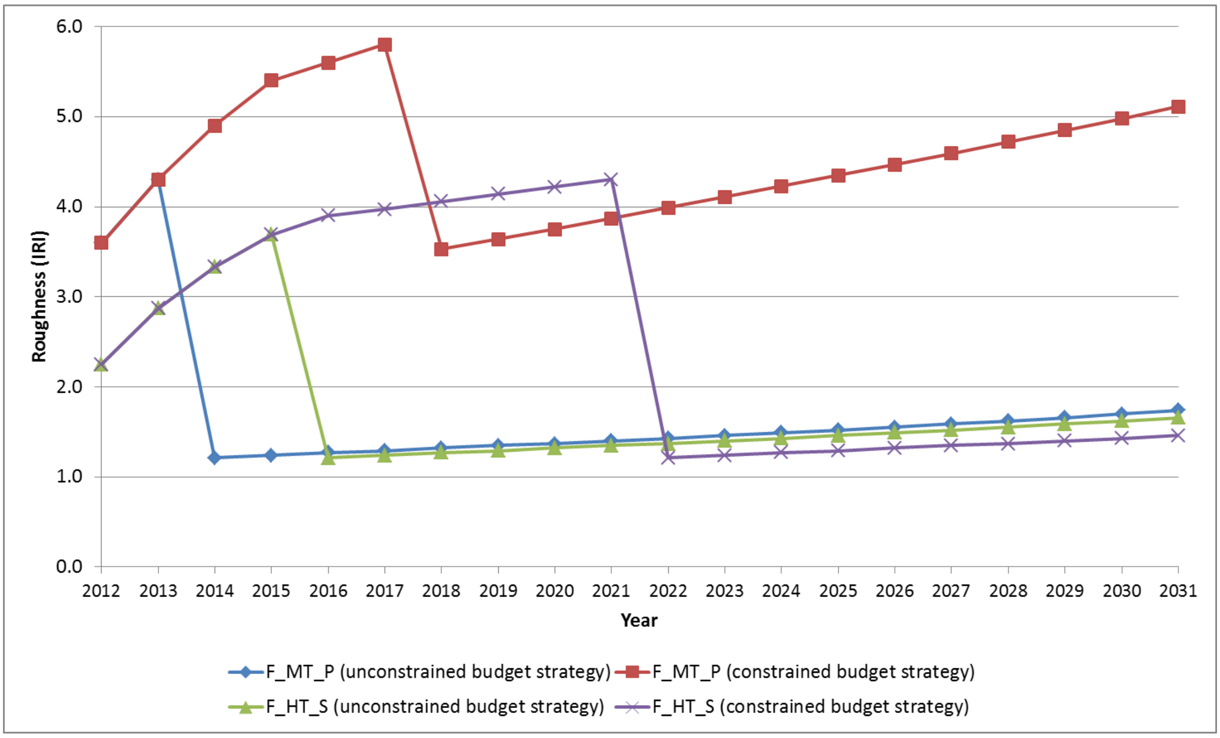

Figure 8.5: Pavement performances for two road groups with the constrained and unconstrained budget strategies

8.3 Importance of an RD Model for a Post-flood Treatment Selection

The new RD models with flooding can be used to predict pavement deterioration after a flood and consequently for treatment selection. For example, with and without flooding scenarios for the road group F_HT_S is shown in *Figure 8.6*. If no flood was assumed, then this road group required OV 30 mm at 4.0 IRI in year 9 which costs at $0.4bn. Otherwise, it needed OV 200 mm at 3.0 IRI in year 4 as an unconstrained budget solution with $1.25bn assuming a flood in year 1. *Figure 8.6* shows that this road deterioration was predicted up to year 1 to 4 using the derived roughness based RD model of this road group. If the RD model with flooding was considered only in year 1 to 2 and normal deterioration in year 2 to 4 was predicted by HDM-4, then a different and wrong treatment would have been chosen; which did not reflect a true deterioration with flooding. Similarly, if the RD model was used between year 1 to 3 and normal deterioration in year 3-4, then a post-flood treatment selection would be correct up to year 3. However, if a treatment is used in year 4, then wrong prediction after a flood in year 3-4 would provide an incorrect treatment selection. These findings are shown clearly in *Table 8.3*. In reality, a wrong treatment is chosen currently because of not having any RD models with flooding. As a result, a new treatment might be selected or the standard (3.0 IRI) may be compromised to accommodate the required budget.

Table 8.3: Use of an RD model with flooding for a post-flood treatment selection for the road group F_HT_S

Scenarios	IRI prediction using RD model with flooding				Remarks
	Year 1	Year 2	Year 3	Year 4	
Normal deterioration (no flood)	2.25	2.31	2.37	2.43	No post-flood treatment was required.
RD model used between year 1-2 and normal deterioration between year 2-4	2.25	2.87	2.93	2.99	If a post-flood treatment is chosen in year 2, then appropriate deterioration is obtained with the RD model. However, if a treatment is used in year 3, then wrong prediction after a flood in year 2-3 would provide an incorrect treatment selection.
RD model used between year 1-3 and normal deterioration between year 3-4	2.25	2.87	3.33	3.39	If a post-flood treatment is chosen in year 3, then appropriate deterioration is obtained with the RD model. However, if a treatment is used in year 4, then wrong prediction after a flood in year 3-4 would provide an incorrect treatment selection.
RD model used between year 1-4	2.25	2.87	3.33	3.69	If a post-flood treatment is chosen in year 2, 3 or 4, then appropriate deterioration is obtained using the RD model with flooding.

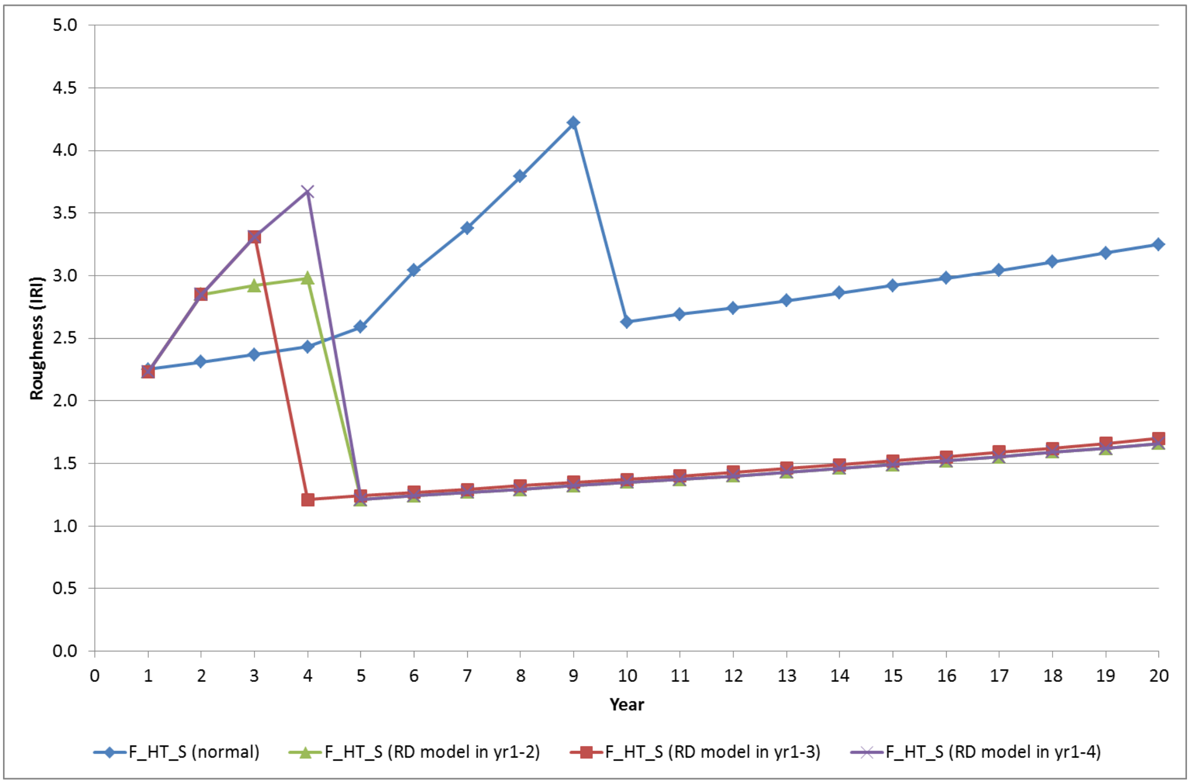

Figure 8.6: Importance of the new RD model with flooding for deterioration prediction and a post-flood treatment selection

8.4 Summary

This chapter discusses the approach used in obtaining the constrained and unconstrained post-flood road maintenance strategies for the TMR-QLD. A flood was assumed in year 1, then the RD models with flooding were used to reflect road deterioration after a flood for up to 2 to 3 years before selecting a post-flood maintenance strategy starting from year 2 using the HDM-4 model. The results reveal that an unconstrained solution will keep the network in excellent to good conditions. However, it needs a huge investment and provides high economic loss. Therefore, the constrained solution obtained using the optimisation objective of 'minimise agency cost at a target IRI' is more realistic. Detailed economic results and pavement performances are shown in this chapter.

CHAPTER NINE: RESULTS OF THE PRE-FLOOD ROAD MAINTENANCE STRATEGY[6]

9.1 Introduction

The detailed approach in deriving a pre-flood road maintenance strategy has been shown in *Figure 3.9*. *Figure 3.1* highlights the components discussed in this chapter. The HDM-4 model was used for this analysis. A treatment was given in year 1 and normal deterioration was predicted until a flood comes. Then, the roughness and rutting based RD models with flooding were used to predict the after flood road deterioration for the first two years after a flood before intervening with the post flood rehabilitation. The remaining life-cycle period was assumed with normal deterioration. There may be more floods, which have not been considered in the current scope of analysis. Moreover, the current approach/framework could be used for two or more flood cases, which was shown in steps I-XI in Section 3.5. The current analysis follows up to steps I-VI covering one flood in life cycle analysis.

It is worth noting that although one flood was assumed in 19 years of analysis after a treatment given in year 1, it did not consider a flooding impact because of a flood with 19 years return period or about 5% probability. The study used a flood to show the usefulness of a pro-active approach in managing roads incorporating flooding. In addition, road deterioration after a flood was not predicted using the RD models at 5% probability of flooding. It would then cause less deterioration and overall pre-flood strategy would be cheaper. This analysis used the RD models at 100% probability of flooding to estimate pavement deterioration which provided the higher ΔIRI and ΔRutting. Therefore, it used the highest after flood impact on pavement performances or the worst case scenario to obtain a safe and sound pre-flood strategy.

The probability of a flood in 19 years with 19 years return period is 0.642; this probability is 0.865, 0.986, 1.000 and 1.000 if return periods are 10, 5, 2 and 1 years respectively. As the TMR-QLD did not have a detailed flood risk map and also as it was a network level analysis, the chance of a flood or more floods are varied. Therefore, the current analysis concentrated

[6] *The findings provided in this chapter has resulted a Journal Paper:* http://ascelibrary.org/doi/abs/10.1061/%28ASCE%29TE.1943-5436.0000901

on at least one flood in life-cycle to get the pro-active pre-flood strategy results. In addition, this research developed the RD models at different probabilities of flooding in order to assess different types of pavement performances and obtaining sound flood-resilient pavements. Therefore, occurrence of flooding was not vital in this analysis; rather the impact of a flood at a certain probability was useful for road deterioration predictions and pre- and post-flood strategy selection.

9.2 Economic Analysis Results

The analysis used both the flexible and composite pavement road groups. The pre-flood treatments to strengthen pavement structural strength were mainly optimum treatments discussed in Chapter Seven. These are thin overlay (generally OV 30 mm) and given in year 1 at $21.13bn. If a flood comes randomly in year 2, 4, 7, 12 or 17, a thin overlay was required as after flood rehabilitation. On the other hand, the post-flood strategy shows that the after flood rehabilitation are mostly thicker overlays. The pre-flood strategy considers a treatment with thin overlay in year 1, which increases pavement strength and makes sure that they perform well after a flood. As a result, after flood rehabilitation under a pre-flood strategy was cheaper and could manage to maintain the network better.

The detailed after flood strategy because of a flood at varying years are given in *Appendix 9A to 9E*, and the summarised results are shown in *Table 9.1*. The after flood rehabilitation costs were almost close when a flood was assumed in any year, and it ranged in between $15.80bn and $17.00bn. As a result, including the initial pre-flood strengthening costs of $21bn, the total pre-flood strategy costs were around $37bn to $38bn. As this strategy suggests strengthening of pavements at the beginning, they could perform better with flooding and needed a thin overlay as rehabilitation. Moreover, if a flood comes late, then normal deterioration would prevail in the years before the flood; and hence the average network condition is better.

The pre- and post-flood strategy results provided five major indicators for comparison, i.e., budget, average network performance in IRI, NPV, NPV/Cost and Benefit/Cost. Although, a pre-flood strategy required $37bn to $38bn to achieve an average IRI of between 4.25 and 5.10, it provided '+ve' economic benefits in life-cycle in between $2.8bn to $6.7bn; and others none. The pre-flood strategy provided positive NPV/Cost for all the scenarios and >1

Benefits/Cost. The unconstrained post-flood strategy could manage roads at 1.50 to 2.00 IRI, but it needed \$49.7bn and provided huge economic loss of \$27.7bn. Similarly, the constrained post-flood strategy could manage the network at 5.20 to 5.90 IRI with \$26bn to \$27bn, but it produced economic loss in all the cases (see *Table 9.2* and *Figure 9.1*). The pre-flood strategy could maintain network better with an average less roughness of 0.93 IRI compared to the post-flood one. Moreover, it provided economic benefits in the life-cycle.

It is worth noting that all the costs and benefits are discounted at base year to get the NPV values. *Table 9.2* reveals NPVs at before and after a flood for different scenarios, ultimately life-cycle NPVs which is also given in *Figure 9.1*. The '+ve' NPVs were found for any investment before a flood at year 1. As all the road groups had several flood affected roads, investment in year 1 certainly provided benefits to these roads as well as a road group in a broader sense. In fact, any investment like the proposed pre-flood strategy reduces IRI, and as a result VOC and travel time savings are occurred which provide positive benefits for any kind of roads.

The analysis assumed a flood in the life-cycle for the pre- and post-flood road maintenance strategy. The main aim was to obtain a pro-active maintenance strategy for a road network that deals with circumstances before and after a flood, which is termed as a pre-flood strategy. On the other hand, if roads are treated after a flood only as a reactive approach, it is considered as a post-flood strategy. There may be more floods, which have not been considered in the current scope of analysis. However, the current approach/framework could be used for two or more flood events.

Table 9.1: Summarised pre- and post-flood strategy results

Strategy		Features	Costs for Pre-flood Treatments in Year 1 ($ bn)	Costs for After Flood Treatments ($ bn)	Total Required Costs ($ bn)	Estimated Network Roughness (IRI)
Pre-flood Strategy	T1_F2_RD3&4_Post3	Treatments in year 1 (T1), a flood in year 2 (F2), RD models used in year 3 & 4 (RD3&4) and post flood treatments start from year 3 (Post3)	21.13	16.06	37.19	5.10
	T1_F4_RD5&6_Post5	Treatments in year 1 (T1), a flood in year 4 (F4), RD models used in year 5 & 6 (RD5&6) and post flood treatments start from year 5 (Post5)	21.13	16.19	37.32	4.90
	T1_F7_RD8&9_Post8	Treatments in year 1 (T1), a flood in year 7 (F7), RD models used in year 8 & 9 (RD8&9) and post flood treatments start from year 8 (Post8)	21.13	15.87	37.00	4.70
	T1_F12_RD13&14_Post13	Treatments in year 1 (T1), a flood in year 12 (F12), RD models used in year 13 & 14 (RD13&14) and post flood treatments start from year 13 (Post13)	21.13	15.82	36.95	4.40
	T1_F17_RD18_Post18	Treatments in year 1 (T1), a flood in year 17 (F17), RD models used in year 18 (RD18) and post flood treatments start from year 18 (Post18)	21.13	16.94	38.07	4.25
Post-flood Strategy	T0_F2_RD3&4_Post3*	No treatments in year 1 (T0), a flood in year 2 (F2), RD models used in year 3 & 4 (RD3&4) and post flood treatments start from year 3 (Post3)	0.00	49.70 (unconstrained budget)	49.70	1.50 to 2.00
	T0_F2_RD3&4_Post3*	No treatments in year 1 (T0), a flood in year 2 (F2), RD models used in year 3 & 4 (RD3&4) and post flood treatments start from year 3 (Post3)	0.00	26.10 (constrained budget)	26.10	5.20
	T0_F4_RD5&6_Post5	No treatments in year 1 (T0), a flood in year 4 (F4), RD models used in year 5 & 6 (RD5&6) and post flood treatments start from year 5 (Post5)	0.00	26.31 (constrained budget)	26.31	5.40
	T0_F7_RD8&9_Post8	No treatments in year 1 (T0), a flood in year 7 (F7), RD models used in year 8 & 9 (RD8&9) and post flood treatments start from year 8 (Post8)	0.00	25.79 (constrained budget)	25.79	5.70
	T0_F12_RD13&14_Post13	No treatments in year 1 (T0), a flood in year 12 (F12), RD models used in year 13 & 14 (RD13&14) and post flood treatments	0.00	25.71 (constrained budget)	25.71	5.80

		start from year 13 (Post13)				
	T0_F17_RD18_Post18	No treatments in year 1 (T0), a flood in year 17 (F17), RD models used in year 18 (RD18) and post flood treatments start from year 18 (Post18)	0.00	27.53 (constrained budget)	27.53	5.90

Table 9.2: Detailed economic results for the pre- and post-flood strategy

Strategy		Pre-flood Treatment Costs ($bn)	Post-flood Treatment Costs ($bn)	Total Costs ($ bn)	NPV (pre-flood) ($ bn)	NPV (post-flood) ($ bn)	Total NPV ($ bn)	NPV/ Cost	Benefits/ Cost
Pre-flood Strategy	T1_F2_RD3&4_Post3	21.13	16.06	37.19	0.79	3.87	4.66	0.13	1.13
	T1_F4_RD5&6_Post5	21.13	16.19	37.32	2.45	1.87	4.32	0.12	1.12
	T1_F7_RD8&9_Post8	21.13	15.87	37.00	5.21	1.49	6.70	0.18	1.18
	T1_F12_RD13&14_Post13	21.13	15.82	36.95	10.59	-6.01	4.58	0.12	1.12
	T1_F17_RD18_Post18	21.13	16.94	38.07	17.13	-14.35	2.78	0.07	1.07
Post-flood Strategy	T0_F2_RD3&4_Post3 (unconstrained)	0.00	49.70	49.70	0.00	-27.70	-27.70	-0.56	0.44
	T0_F2_RD3&4_Post3 (constrained)	0.00	26.10	26.10	0.00	-7.10	-7.10	-0.27	0.73
	T0_F4_RD5&6_Post5 (constrained)	0.00	26.31	26.31	0.00	-6.40	-6.40	-0.24	0.76
	T0_F7_RD8&9_Post8 (constrained)	0.00	25.79	25.79	0.00	-5.25	-5.25	-0.20	0.80
	T0_F12_RD13&14_Post13 (constrained)	0.00	25.71	25.71	0.00	-3.54	-3.54	-0.14	0.86
	T0_F17_RD18_Post18 (constrained)	0.00	27.53	27.53	0.00	-1.46	-1.46	-0.05	0.95

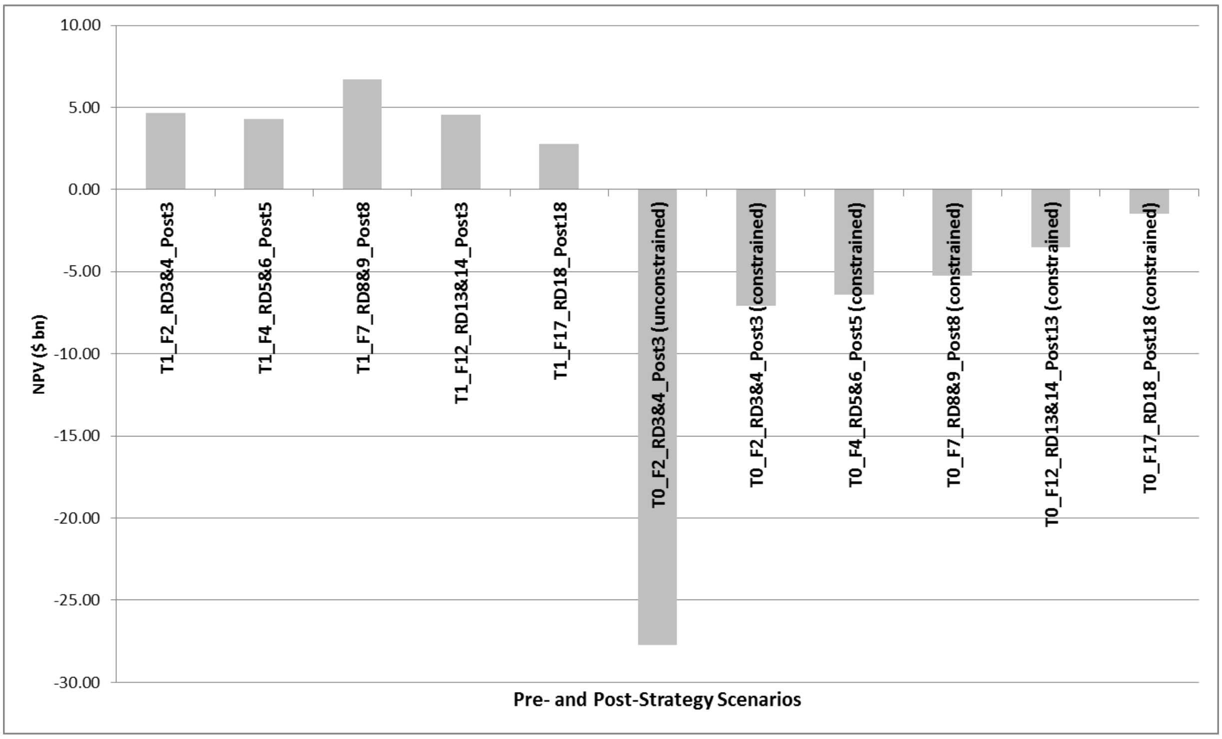

Figure 9.1: NPVs obtained at different pre- and post-flood strategy scenarios

9.3 Pavement Performances with the Pre-flood Strategy

Life-cycle performances of the whole network, especially flexible and composite road groups are given in *Figures 9.2 to 9.6*. Here, the F_LT_P means Flexible pavement with Low Traffic loading and Poor strength road, F_MT_F means Flexible pavement with Medium Traffic loading and Fair strength road, and C_HT_S means Composite pavement with High Traffic loading and Strong strength road groups. Pavement life-cycle performances at a scenario of pre-flood treatments in year 1, flooding in year 2, RD models used in year 3 and 4, and post flood treatments starting from year 3 (T1_F2_RD3&4_Post3) is shown in *Figure 9.2*. Similarly, *Figures 9.3 to 9.6* represent pavement performances with a flood in year 4, 7, 12 and 17 respectively. All these show a normal deterioration before a flood comes, then a sudden increase in roughness was estimated using the RD models with flooding for 2 years due to a flood. The after flood rehabilitation improves road condition. Finally, normal deterioration continues for the remaining period.

After a pre-flood treatment, the average network roughness was around 2.25 IRI and the normal deterioration in the next 15 years went up to 3.75 IRI. It reveals an incremental increase in roughness for the network of about 0.10 IRI per year, which is a realistic one.

Recently, Austroads (2015) observed yearly roughness increase of 0.08 to 0.09 IRI for three roads in Australia. Therefore, no treatments were needed in this period.

The average network performances with treatment in year 1 and at any years due to a flood in year 2, 4, 7, 12 or 17 are shown in *Figure 9.7*.

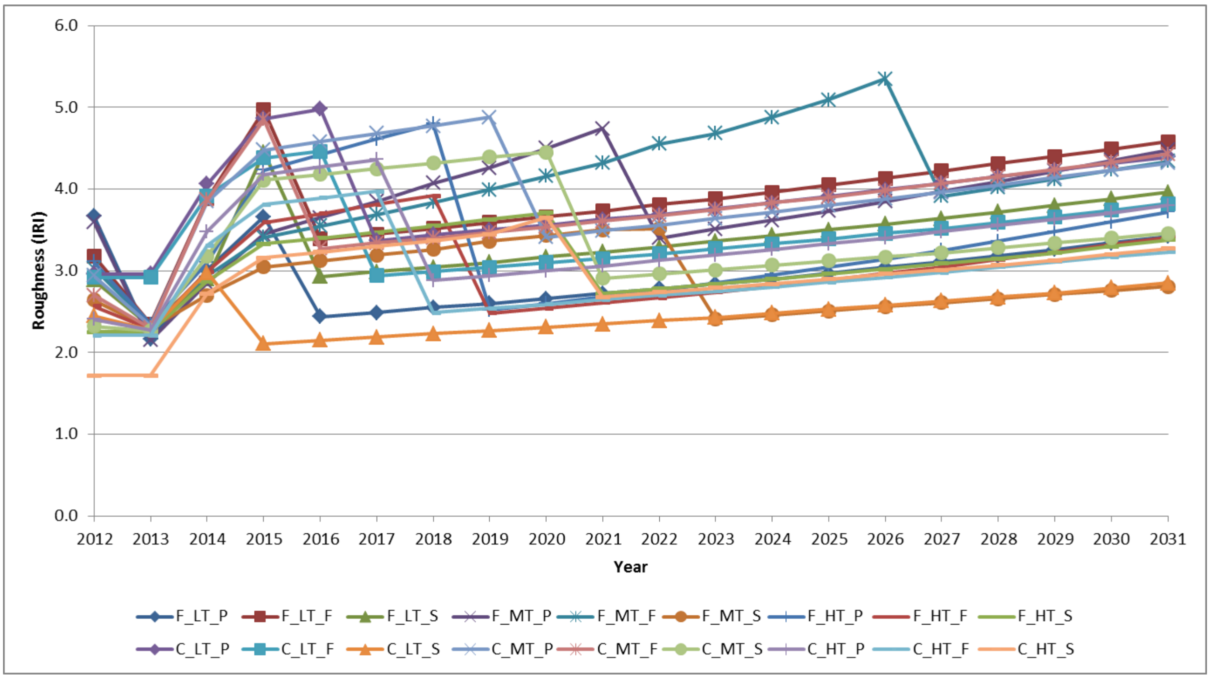

Figure 9.2: Life-cycle performances for all the flexible and composite road groups at treatments in year 1 (2012) and flood in year 2 (2013)

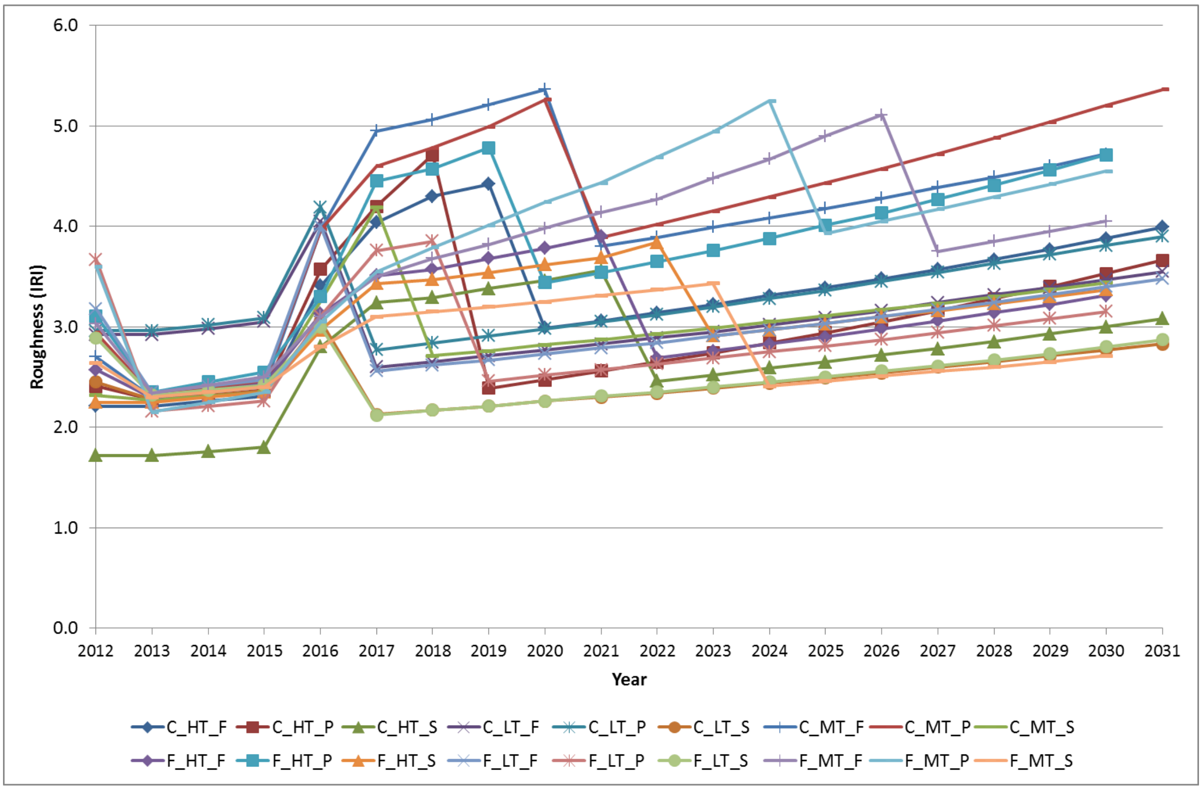

Figure 9.3: Life-cycle performances for all the flexible and composite road groups at treatment in year 1 (2012) and flood in year 4 (2015)

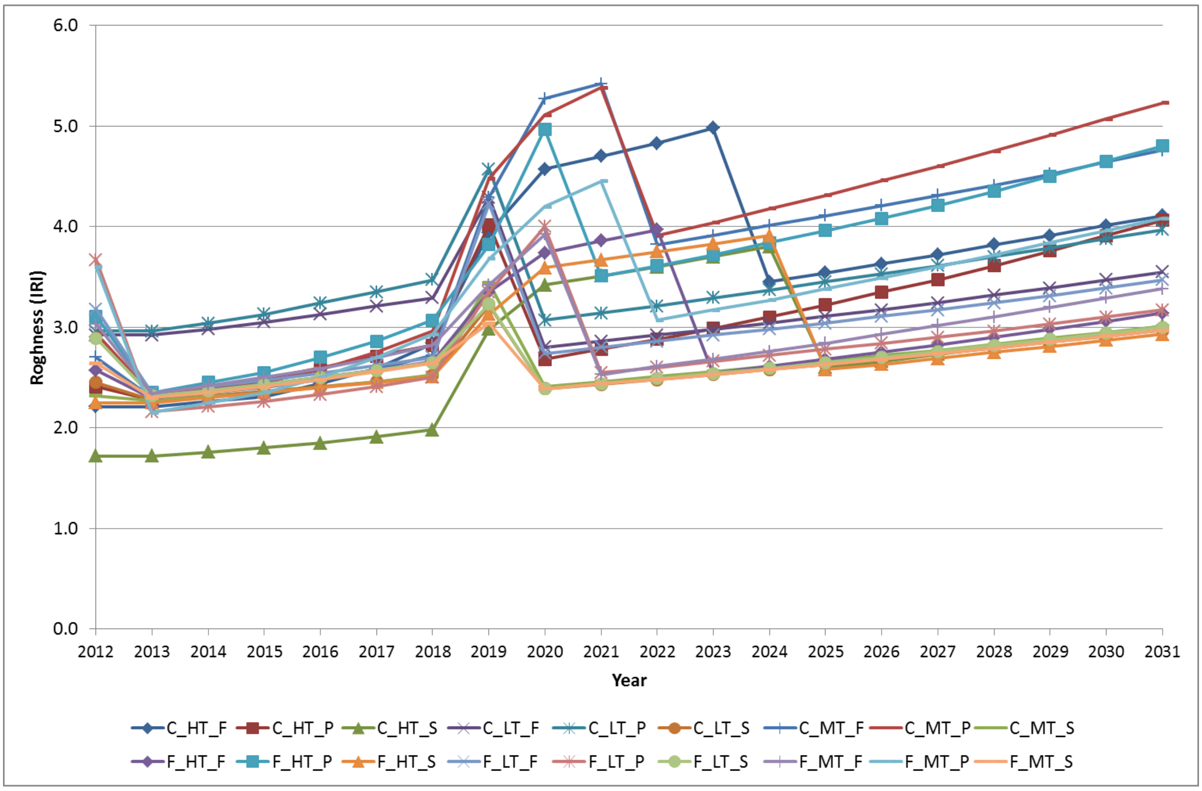

Figure 9.4: Life-cycle performances for all the flexible and composite road groups at treatment in year 1 (2012) and flood in year 7 (2018)

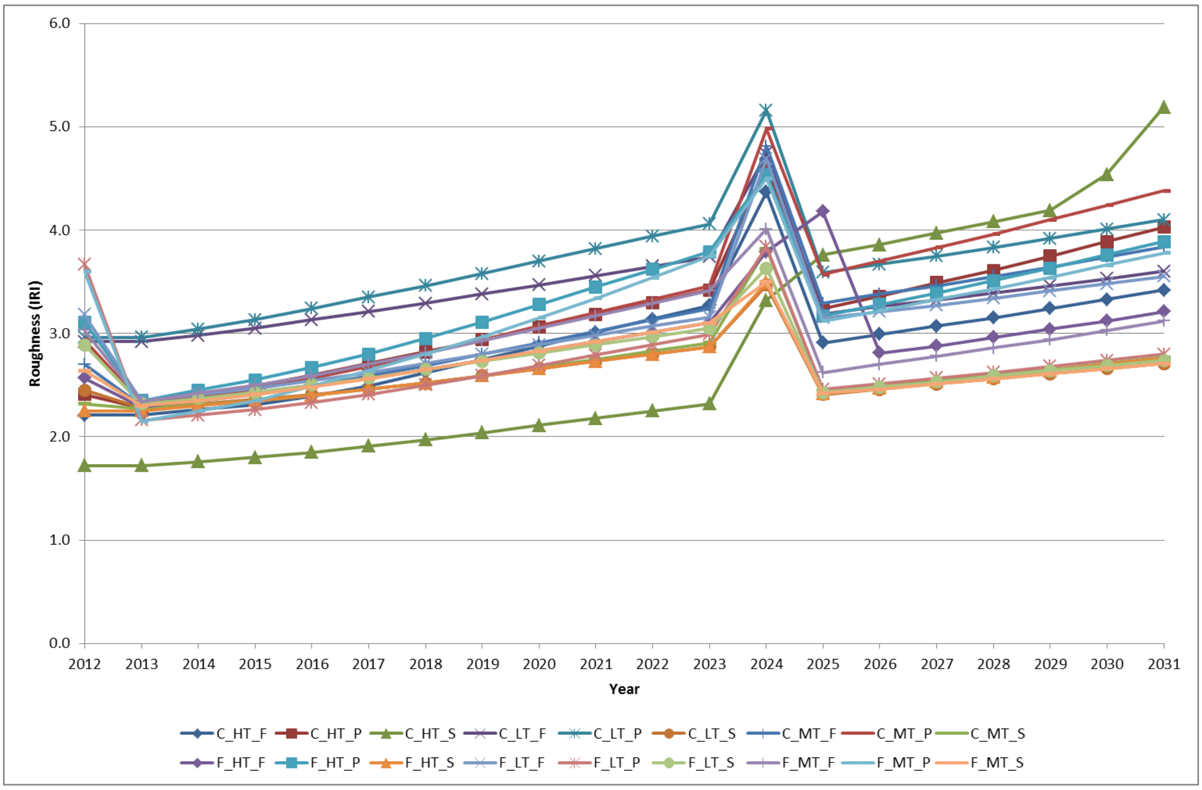

Figure 9.5: Life-cycle performances for all the flexible and composite road groups at treatment in year 1 (2012) and flood in year 12 (2023)

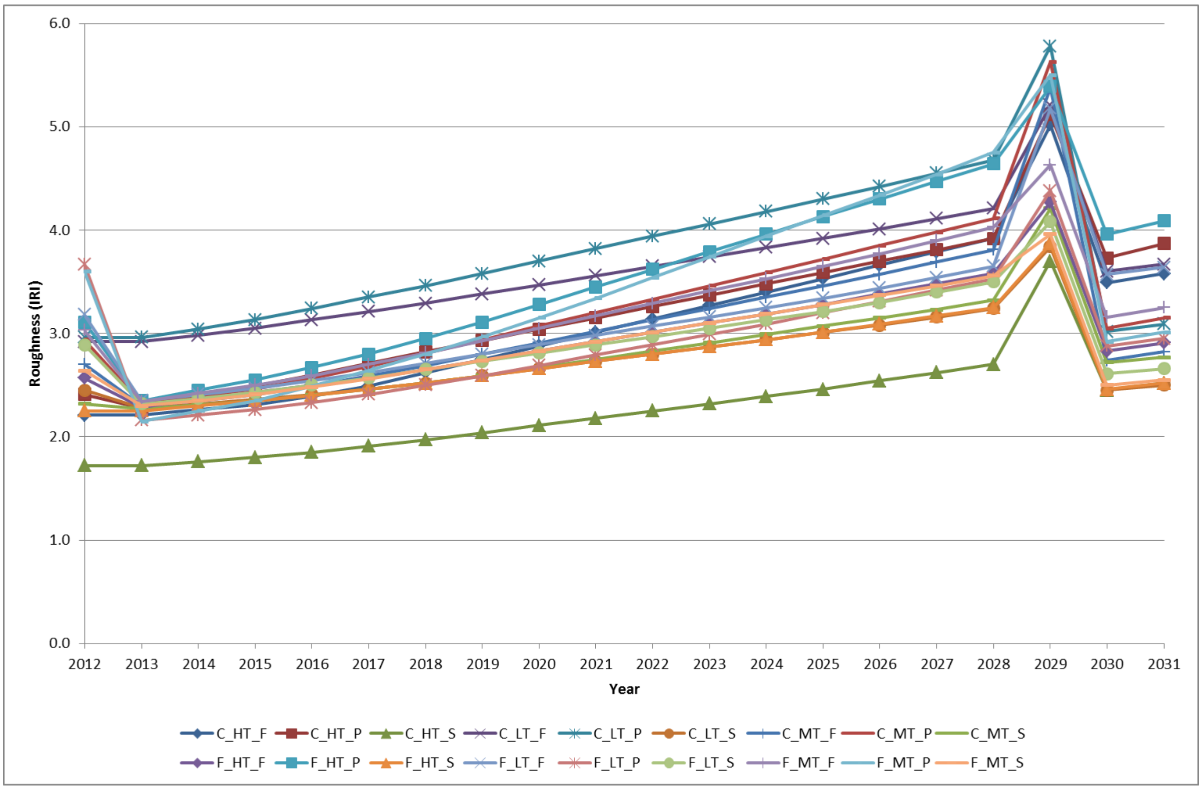

Figure 9.6: Life-cycle performances for all the flexible and composite road groups at treatment in year 1 (2012) and flood in year 17 (2028)

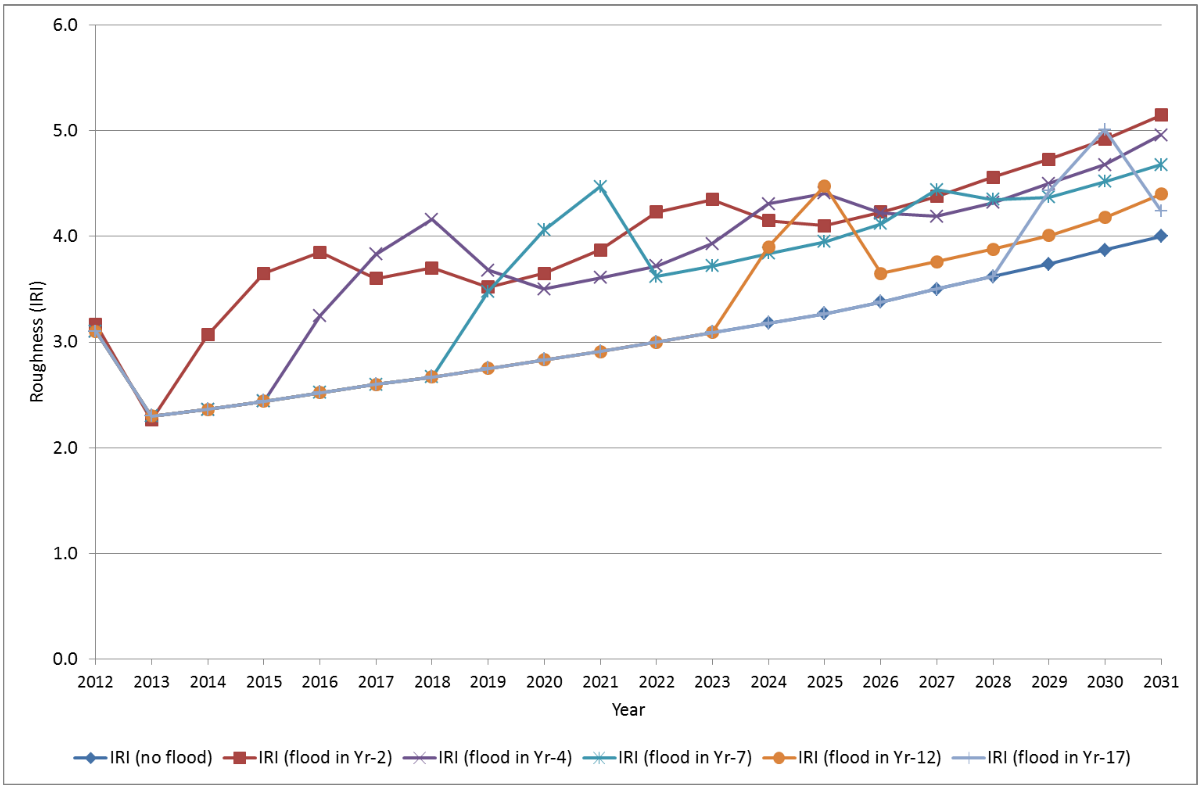

Figure 9.7: Average network performances at treatment in year 1 and flood in different years

9.4 Comparison of the Derived Pre- and Post-flood Strategy

Details of the post-flood strategy results are discussed in the previous chapter. *Section 9.2* provides a comparison of results between the pre- and post-flood strategy (see *Tables 9.1 and 9.2 and Figure 9.1*). More results and comparison on pavement performances are shown here. *Table 9.3* below shows the outcomes on costs/km for different scenarios and IRI variation in the life-cycle. It was discussed earlier that a pre-flood strategy is economically beneficial and it can also keep a road network at a higher standard. It shows that if roads are strengthen now before a flood, it would require less funds for after flood rehabilitation, and also can maintain the network better with a flood. Therefore, the pre-flood strategy is useful and effective.

It is worth noting that if a flood is delayed, the roughness before post-flood treatments gets higher due to road deterioration with time. As a result, the hypothetical scenario of pre-flood treatments in year 1, flooding in year 17, RD models used in year 18, and post flood treatments starting from year 18 (T1_F17_RD18_Post18) needs the highest cost and a thick overlay for some road groups.

The average network life-cycle performances with the optimum, pre- and post-flood strategies are shown in *Figure 9.8*. The network performances with the pre-flood and without pre-flood (with post-flood) strategies are given in *Figure 9.9*. As an example, similar pavement performances are shown with the optimum, pre- and post-flood strategies for the Flexible pavement with High Traffic loading and Strong strength road (F_HT_S) and Flexible pavement with Medium Traffic loading and Poor strength road (F_MT_P) road groups (see *Figures 9.10* and *9.11* respectively).

In all cases, a pre-flood strategy provides a better network life-cycle performance (see *Figure 9.8*). Comparing results help a road authority getting a sound decision in investment to tackle a flood. The pre-flood strategy is an innovative approach by upgrading all road groups now with a thin overlay so that they perform well with flooding in future. It would also cost less for an after flood rehabilitation, although an initial investment is required. It is worth noting that an unconstrained budget can keep a road network at best condition, but a constrained budget does not suggest strongest treatments and can keep a network at good to fair condition.

Table 9.3: Summarised pre- and post-flood maintenance strategy results

Strategy	Scenario	Total Costs ($ bn)	Costs /km ($ bn/km)	ΔIRI for Post Flood Treatment	Life-cycle Avg. Roughness (IRI)
Optimum strategy	Without a flood in life-cycle	17.79	0.00075	-	4.00
Post-flood strategy	T0_F2_RD3&4_Post3 (unconstrained budget)	49.70	0.00210	2.67 IRI	1.50 to 2.00
	T0_F2_RD3&4_Post3 (constrained budget)	26.10	0.00110	0.27 IRI	5.20
Pre-flood strategy	T1_F2_RD3&4_Post3	37.19	0.00157	0.15 IRI	5.10
	T1_F4_RD5&6_Post5	37.32	0.00158	0.52 IRI	4.90
	T1_F7_RD8&9_Post8	37.01	0.00156	0.85 IRI	4.70
	T1_F12_RD13&14_Post13	36.95	0.00156	0.88 IRI	4.40
	T1_F17_RD18_Post18	38.07	0.00161	0.77 IRI	4.25

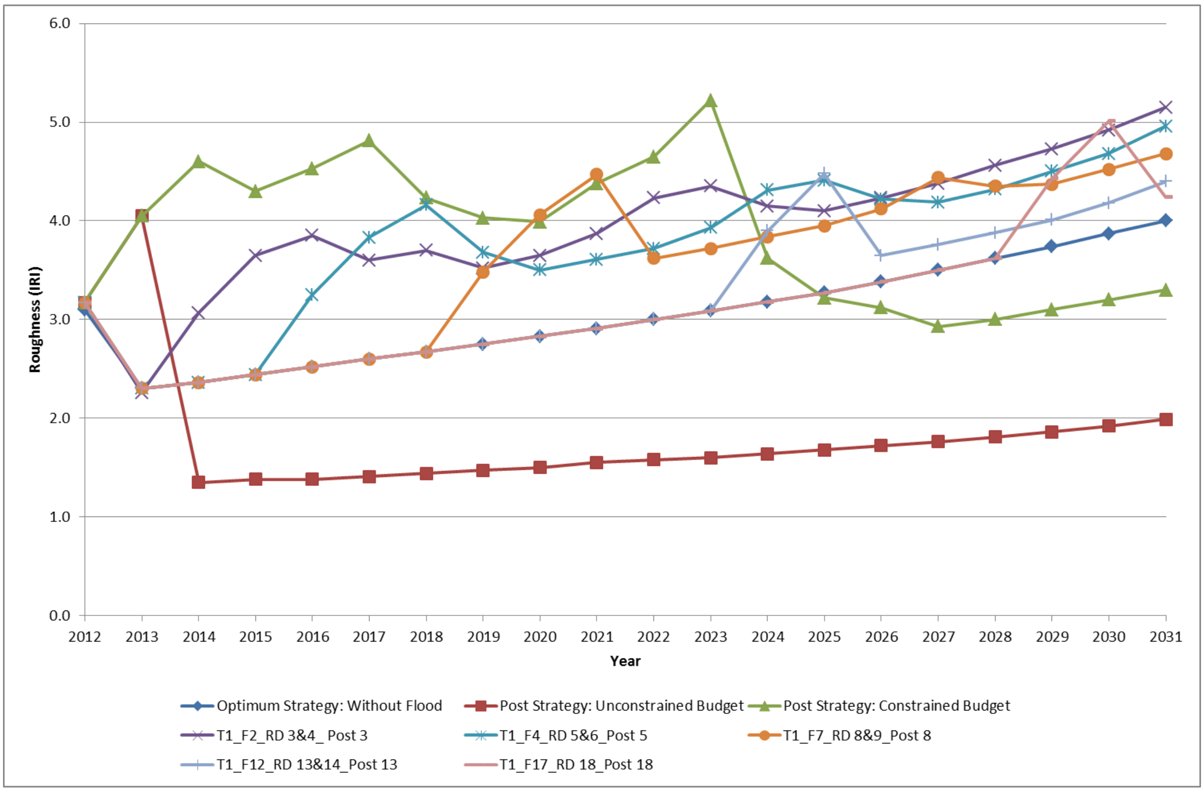

Figure 9.8: Average network life-cycle performances at the optimum, pre- and post-flood strategies

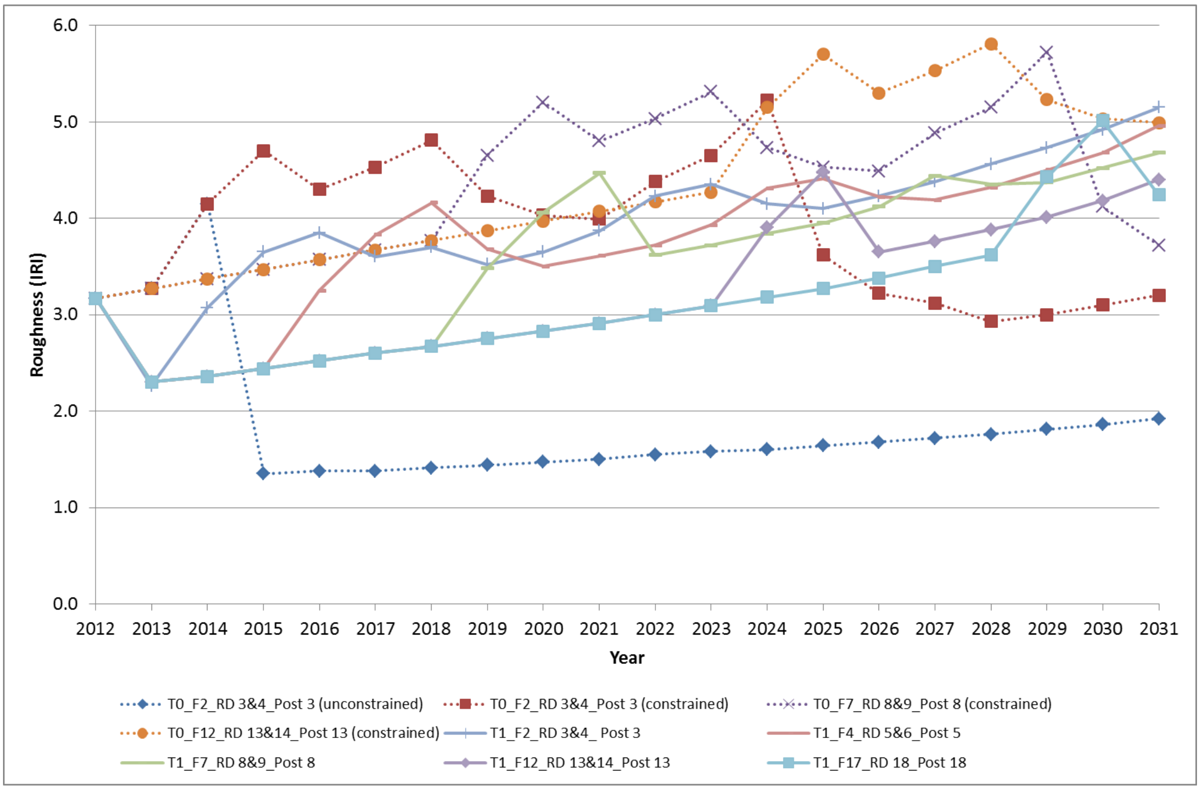

Figure 9.9: Average network life-cycle performances with the pre-flood and post-flood strategies

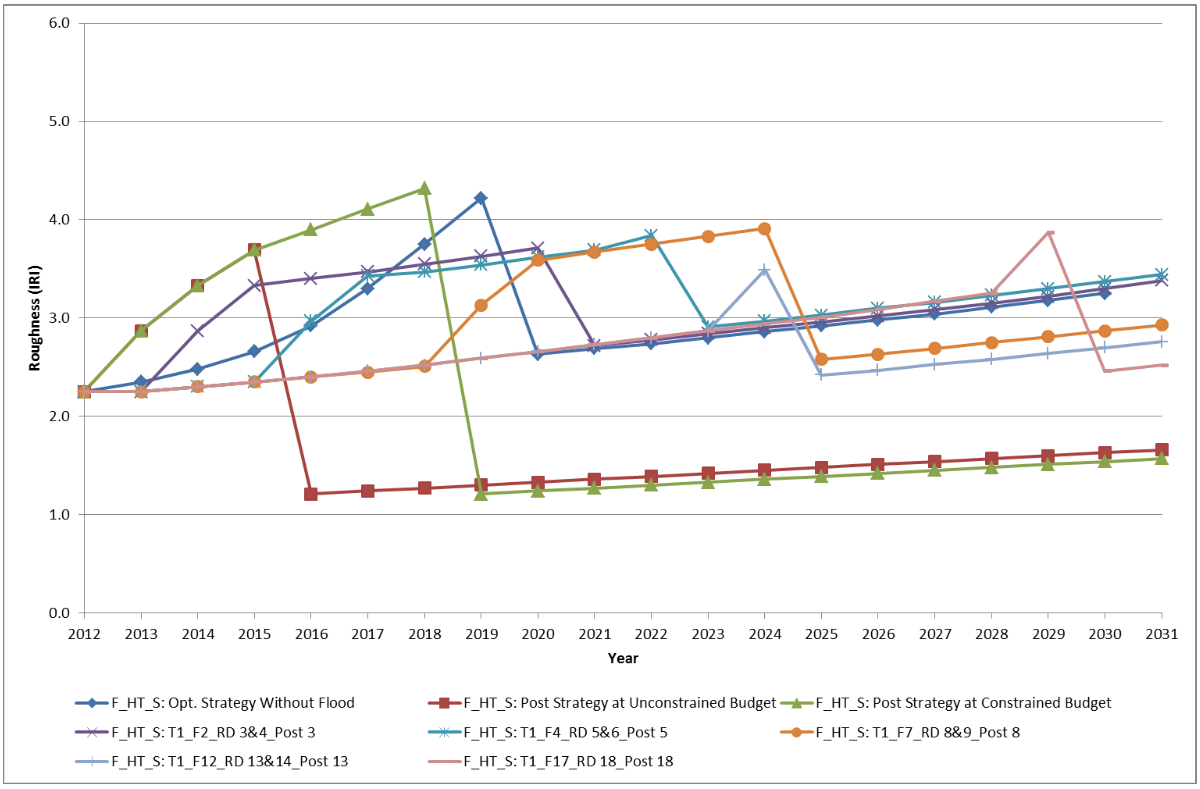

Figure 9.10: Pavement life-cycle performances at the optimum, pre- and post-flood strategies for the F_HT_S road group

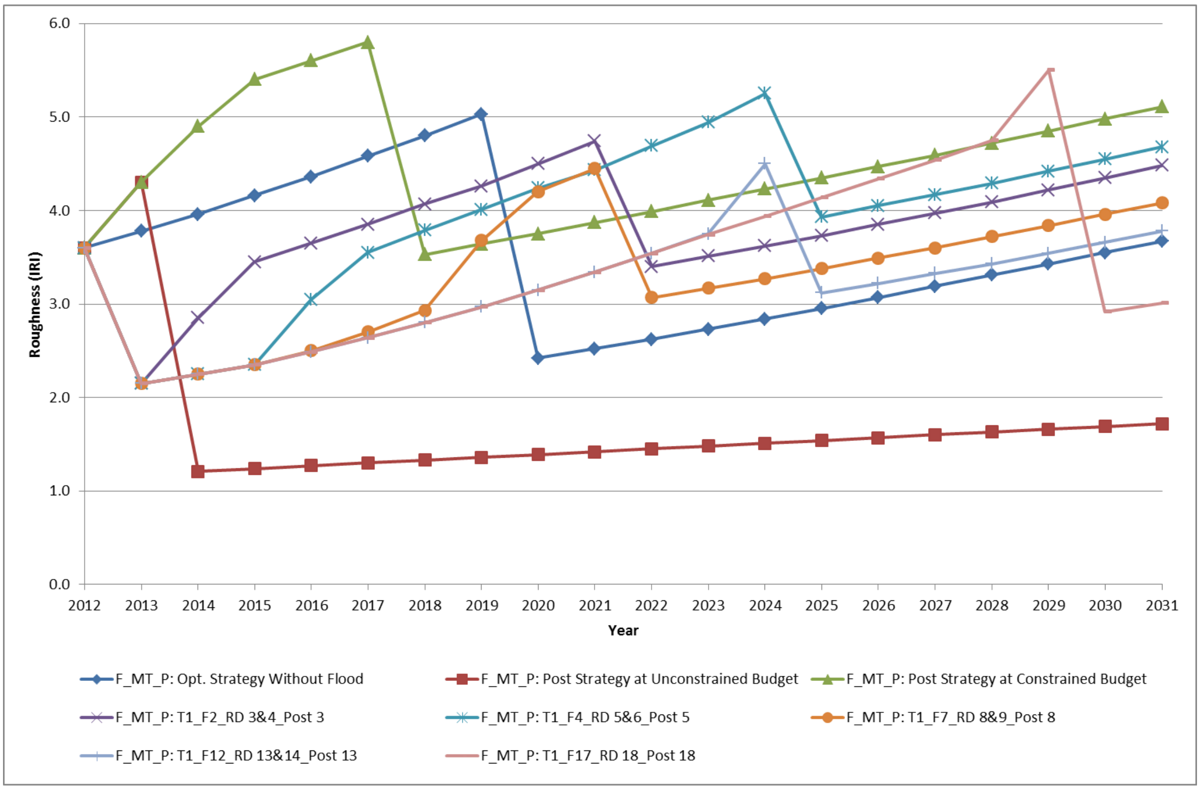

Figure 9.11: Pavement life-cycle performances at the optimum, pre- and post-flood strategies for the F_MT_P road group

9.5 Summary

This chapter shows the pre-flood road maintenance strategy results. Treatments were assumed in year 1 which was basically a thin overlay to increase pavements strength. As a result, a road performs well if a flood comes at any year. The new RD models with flooding were used after a flood for predicting road deterioration for two years. The after flood treatments, chosen from the first year after a flood, were also included in the pre-flood strategy. The usefulness of applying a pre-flood strategy has been shown through comparing results with the post-flood strategy. Finally, pavement performances at different strategies and scenarios are also shown.

CHAPTER TEN: RESULTS OF THE FLOOD RISK ASSESSMENT[7]

10.1 Introduction

This chapter shows results of the flood risk analysis done for the whole road network of TMR-QLD. The approach was highlighted in Section 3.7. The components discussed here are shown in *Figure 3.1*.

The roughness vs. time data for each road group indicated an abrupt increase in roughness due to any flood event, which helped to obtain flooding probability values from the two flood events. In the analysis, an assumption using engineering judgment has been made about the flooding probability when two abrupt IRI changes were not observed for some road groups, although about 10-12 years of IRI vs. time data were used. The weighted average IRI values from roughness distribution data before and after a flood were used to calculate the flooding consequences using the *Equation 3.4* for a road group. Finally, the risk scores were obtained using the *Equation 3.5* from the likelihood and consequences values.

10.2 Main Results

The detailed flood risk scores for all road groups are given in *Table 10.1*. An example of the above calculation has been shown below for a road group of Flexible pavement with Low Traffic loading and a Fair strength (F_LT_F). The flooding frequency/return period was observed >10 years because of not having some more years' data; therefore, the likelihood was in between 'moderate' to' rare'. Considering the frequency of flooding in Queensland and most of the roads' data, a flood frequency of 20 years was assumed.

Hence, Likelihood score = 2.75;
Consequence was estimated based on the roughness distribution before and after the flood;
Average after flood IRI = 3.03 m/km (determined from the after flood roughness distribution);
Average before flood IRI = 1.00 m/km (not shown here);
Consequence = (3.03 – 1.00) = 2.03 IRI;

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[7] *The findings provided in this chapter has resulted a Journal Paper:* http://www.icevirtuallibrary.com/doi/abs/10.1680/jtran.15.00120

Risk score = 2.75 * 2.03 = 5.58; and
Risk zoning = 'moderate'.

The F_LT_F road group was at 'moderate' risk zone with a risk score of 5.58. The results showed that generally the strong strength with high or medium traffic loading road groups had low consequences, and low risk scores. For example, the Flexible pavement with Medium Traffic loading and Strong strength (F_MT_S), Flexible pavement with High Traffic loading and Strong strength (F_HT_S), Composite pavement with Medium Traffic loading and Strong strength (C_MT_S) and Rigid pavement with High Traffic loading and Strong strength (R_HT_S) provided low consequence scores of 0.22, 0.24, 0.41 and 0.48 respectively. As a result, they had low risk scores of 0.77, 1.12, 1.39 and 2.40. If the flood consequence of a road estimated from before and after a flood was small, it indicates that the road deteriorated less after a flood compared to its normal deterioration and might be considered as a flood resilient pavement. As a result, the above road groups of F_MT_S, F_HT_S, C_MT_S and R_HT_S were all flood resilient. Apart from that, there were four other road groups who had low consequence score of less than 0.50 IRI, and might be considered as flood resilient pavements.

Most of the rigid pavement road groups had higher consequences than the flexible pavements using the same flooding probability and they were in the critical risk zone. This result should be reviewed with caution. This was because of the assumption of a flood based on the roughness jump, initial IRI, and roughness distribution with and without flooding. Moreover, the rigid pavement road groups had very limited road data, and hence length weighted average IRI distribution data before and after a flood also affect the result. Intuitively, a rigid pavement performed well after a flood, which was shown in Chapter Six.

Different probability of a flood with changed initial conditions may provide different risk results. As flooding probability was not the same for different road groups, and different flooding probabilities might have different consequences, a thorough investigation has been carried out for the road groups with the same flooding probability to assess their consequences. This would give a clear picture on pavement responses at the same probability.

For example, road groups with a flooding probability of 1 in 2 years (50%) have been reviewed. It appears that a strong road performed better than a fair road of the same pavement type and loading, a poor road performed the worst. For example, the R_HT_S had a consequence score of 0.48, and the Rigid pavement with High Traffic loading and Poor strength (R_HT_P) had a consequence score of 0.93. The consequence score for R_HT_F was not comparable, as its probability of flooding was not 50%.

A high loading road group has higher standards and receives an appropriate attention with adequate maintenance, which ensures a better performance. As a result, at the same flooding probability, the R_HT_S had a consequence score of 0.48 and Rigid pavement with Medium Traffic loading and Strong strength (R_MT_S) had 1.13. Moreover, at the flooding probability of 1 in 8 years (12.5%), a Flexible pavement with Medium Traffic loading and Fair strength (F_MT_F) had a consequence score of 0.21 and a Composite pavement with Medium Traffic loading and Fair strength (C_MT_F) had a score of 0.86, which indicated that a flexible pavement performed better than a composite road group. A composite pavement stabilized materials may break after some years, and then it works like a flexible pavement; which may affect the performance result.

However, a detailed field investigation is needed in future, as a composite pavement generally performs better than a flexible one. A complete flood risk assessment is required, as different flooding probabilities have different consequences on roads' performances. A consequence score may change even at the same probability of flooding for a same road because of different initial road conditions.

The above risk scores, zoning and consequences, and analyses at the same probability provided an indication of flood-resilient pavements. It was concluded that a strong pavement with high traffic loading at a higher standard performs the best after a flood. These results devise an efficient method in upgrading the flood damaged pavements into flood-resilient pavements by increasing their strength. It can be done by stabilising granular and subgrade layers and/or increasing asphalt concrete thickness to enhance a pavement's structural strength. Generally, a flood resilient pavement performs better if a flood comes in the life-cycle and needs thinner overlay than a post flood treatment (see Chapter Nine).

While investigating pavement performances in year 1 after a flood, it has been found that a strong and rigid pavement with high loading at high standard could perform well (see Chapter Six). The two new indicators, i.e., ΔIRI/Pr and ΔIRI/MrL, provided results on pavements flood resilience. These results were consistent with the current one, apart from a rigid pavement issue.

Table 10.1: Flood-risk assessment results for all the road groups

Road Groups	Frequency of Flooding	Likelihood of Flooding Score	Roughness Distribution After the Flood (to get average after-flood IRI)					Consequence Score (average IRI after a flood – average IRI before the flood)	Risk Score	Risk Zoning
			<2.0 IRI	2.0 -3.0 IRI	3.0 – 4.0 IRI	4.0 – 5.0 IRI	>5.0 IRI			
F_LT_P	> 10 years	2.75	1%	19%	35%	26%	19%	0.62	1.71	Low
F_LT_F	> 10 years	2.75	10%	42%	32%	14%	3%	2.03	5.58	Moderate
F_LT_S	10 years	3.00	11%	44%	26%	13%	5%	0.56	1.68	Low
F_MT_P	> 10 years	2.75	3%	29%	38%	24%	5%	0.66	1.82	Low
F_MT_F	8 years	3.40	13%	44%	28%	12%	3%	0.21	0.71	Low
F_MT_S	> 5 years	3.50	20%	50%	20%	7%	3%	0.22	0.77	Low
F_HT_P	9 years	3.20	15%	38%	30%	13%	4%	0.38	1.22	Low
F_HT_F	2 years	5.00	26%	43%	18%	10%	3%	0.25	1.25	Low
F_HT_S	3 years	4.67	35%	53%	11%	1%	1%	0.24	1.12	Low
C_LT_P	> 9 years	3.00	0%	14%	29%	43%	14%	1.50	4.50	Moderate
C_LT_F	> 5 years	3.50	0%	50%	38%	0%	13%	0.67	2.35	Low
C_LT_S	> 10 years	2.75	34%	16%	41%	8%	0%	0.43	1.18	Low
C_MT_P	9 years	3.20	0%	36%	36%	27%	0%	0.68	2.18	Low
C_MT_F	8 years	3.40	20%	40%	35%	3%	2%	0.86	2.92	Low
C_MT_S	> 6 years	3.40	37%	36%	17%	9%	2%	0.41	1.39	Low
C_HT_P	2 years	5.00	11%	67%	19%	4%	0%	0.59	2.95	Low
C_HT_F	> 10 years	2.75	11%	14%	40%	34%	2%	1.15	3.16	Low
C_HT_S	6 years	3.80	23%	77%	0%	0%	0%	0.69	2.62	Low
R_LT_P	> 10 years	2.75	0%	0%	100%	0%	0%	2.50	6.88	Moderate
R_LT_F	5 years	4.00	0%	5%	35%	35%	25%	1.03	4.12	Moderate
R_LT_S	> 5 years	3.50	0%	0%	0%	100%	0%	1.51	5.29	Moderate
R_MT_P	2 years	5.00	0%	33%	33%	33%	2%	1.03	5.15	Moderate
R_MT_F	2 years	5.00	0%	54%	27%	6%	12%	0.70	3.50	Low
R_MT_S	2 years	5.00	25%	42%	33%	0%	1%	1.13	5.65	Moderate
R_HT_P	2 years	5.00	0%	1%	0%	0%	99%	0.93	4.65	Moderate
R_HT_F	> 3 years	4.25	0%	50%	50%	0%	0%	1.13	4.80	Moderate
R_HT_S	2 years	5.00	61%	36%	3%	0%	0%	0.48	2.40	Low

*As an example, F_LT_F means a road group comprising Flexible pavement with Low Traffic loading and Fair strength; C_MT_P means a road group comprising Composite pavement with Medium Traffic loading and Poor strength; and R_HT_S means a road group comprising Rigid pavement with High Traffic loading and Strong strength
**See *Table 3.2* for risk zoning

The *Figures 10.1 and 10.2* provide comparative results in obtaining flood-resilient pavements by using the ΔIRI/Pr, ΔIRI/MrL, flood-risk consequences and risk scores before and after a flood. The main difference between these two Figures is: *Figure 10.1* provides result with risk score and *Figure 10.2* with consequence score. The lower is the ΔIRI/Pr or ΔIRI/MrL, the more a pavement is flood-resistant. On the other hand, the risk scores indicated that nine road groups were at critical risk zones having scores between 4.12 and 6.88, mainly at lower boundary of the 'moderate' risk zone. Moreover, their consequence scores (or ΔIRI) before and after a flood were also higher with over 0.90 IRI. It indicated that these roads performed poorly compared to the others.

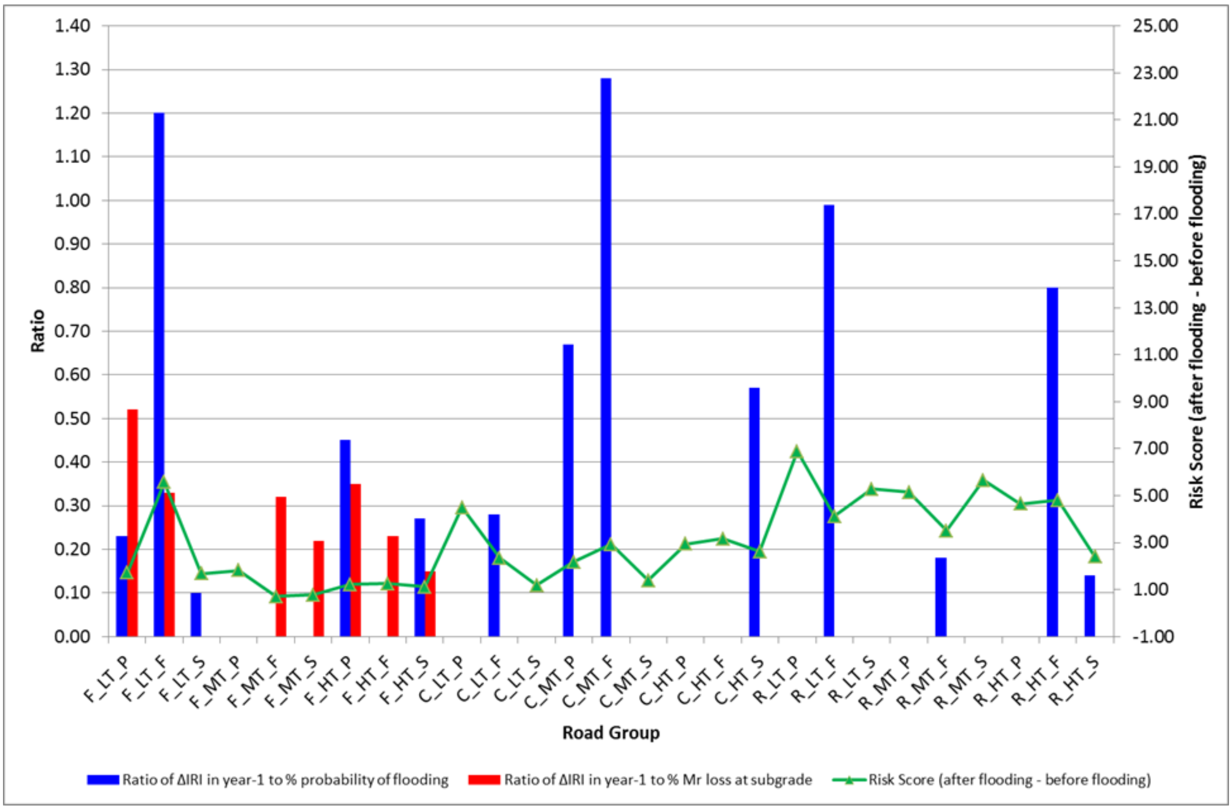

Figure 10.1: Obtaining flood-resilient pavements using the ΔIRI/Pr, ΔIRI/MrL and risk scores before and after a flood

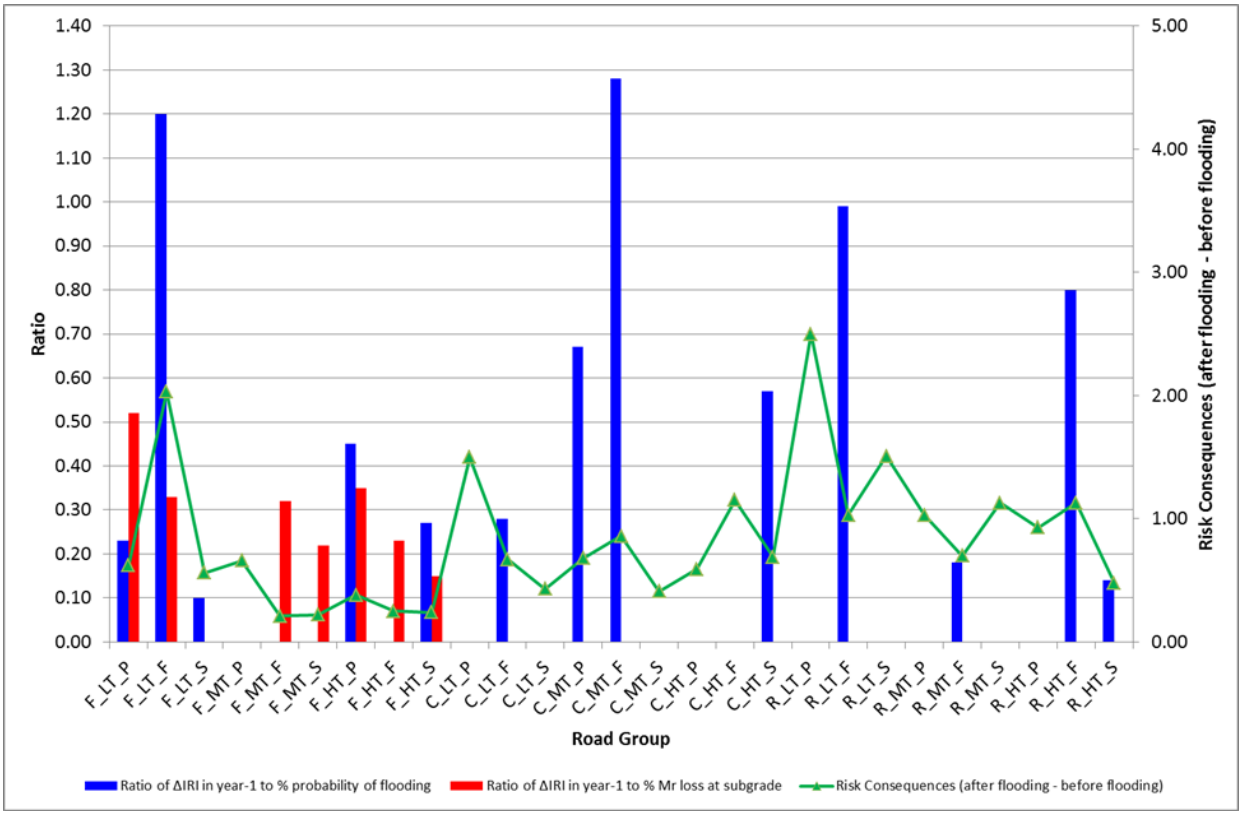

Figure 10.2: Obtaining flood-resilient pavements using the ΔIRI/Pr, ΔIRI/MrL and risk consequences before and after a flood

A validation of the risk consequences and risk scores were done for two roads in Logan, Queensland, namely the Mount Lindesay Highway and Beaudesert-Beenleigh road under the authority of TMR-QLD. They represent road group of F_MT_S and Composite pavement with High Traffic loading and Strong strength (C_HT_S) respectively. The January 2011 after flood data were available in the road database. There was 20% probability (1 in 5 years) of flood as shown in the Bureau of Meteorology, Australia report (BoM, 2014). It was found that about 0.45 km of Mount Lindesay Highway and 0.40 km of Beaudesert-Beenleigh road was affected by the flood. The detailed results are shown in *Table 10.2*.

As an example, the Mount Lindesay Highway (F_MT_S) had a flood likelihood of 4.00. The actual ΔIRI or consequence observed at site was 0.12, whereas 0.22 was found for the road group (see *Table 10.2*). The site specific risk score was 0.48, while risk score for the road group was 0.88. As a result, an 83% variation was found. However, both the Mount Lindesay Highway and road group F_MT_S had small flood consequences, and they were at a 'low' risk zone.

Although, there were some variations between the actual flood consequences and risk scores and road group results, both the cases showed that the ultimate risk zones were the same, i.e., ‘low’. The actual results using the January 2011 flood show lower consequences and risk scores than the representative road group results where different floods and initial conditions were used. Therefore, the results had some variations. However, considering the flood and initial road conditions for the road groups which effect on the consequence and risk results, these validations may be believed reasonable. Moreover, the risk scores calculated by the proposed method involve a probability of occurrence which by definition means they do not happen every time. It was expected that the actual and calculated risks averages over many occurrences would be similar.

Table 10.2: Validation of the risk consequences and Scores

Road Name and Group	**Actual Likelihood Score**	**Actual Consequence (after– before flooding)**	**Risk Consequence Score for the Road Group (see *Table 10.1*)**	**Variation of Consequence using Actual vs. Road Group Results**	**Actual Risk Score**	**Risk Score for the Road Group (see *Table 10.1*)**	**Variation of Risk Score using Actual vs. Road Group Results**
Mount Lindesay Highway (F_MT_S)	4.00 (Likely)	0.12 IRI (3.31 – 3.19)	0.22 IRI	83%	0.48	0.77	60%
Beaudesert-Beenleigh Road (C_HT_S)	4.00 (Likely)	0.48 IRI (2.10 – 1.62)	0.69 IRI	44%	1.92	2.62	36%

10.3 Summary

This chapter has shown flood risk analysis results. The likelihood of a flood and its consequences using the ΔIRI values were used to estimate risks. Two road groups’ (network level) risk results were verified with the two site-specific roads actual risk scores and zoning, and found reasonable. Generally, the risk analysis results show that a strong pavement with high traffic loading at a higher standard performs the best after a flood. The analysis revealed that rigid pavements are at risk, which requires further investigation. Detailed pavement performance results after a flood were used to get the flood resilient pavements (shown in Chapter Six), which showed that in reality, a rigid pavement performed the best compared to other two pavement types. The risk consequence scores are also vital along with RD modelling results, ΔIRI/Pr and ΔIRI/MrL to estimate pavement performances after a flood. Therefore, these risk results helps quantifying pavements’ flood resilience.

CHAPTER ELEVEN: CONCLUSIONS AND FURTHER RESEARCH

This chapter discusses the key findings and main contributions to the overall objectives. A recommendation is also included for future research.

11.1 Importance of the Research

Natural disasters may severely affect road infrastructure and accelerate the deterioration of pavements. This thesis focuses on a flooding event which may significantly weaken pavement structures. It is essential to select appropriate treatments for after-flood rehabilitation. Therefore, a pavement performance with flooding is an important element in a PMS. It was found that a few deterioration prediction models addressed flooding, which were regression based and deterministic models. They did not consider probabilistic models to reflect uncertainty of pavement performances. In addition, these models provided deterioration for a short period only; however, in reality a post-flood treatment selection and its implementation need time. These studies did not consider developing network level RD models with flooding. Moreover, no simulation was done for estimating road deterioration at different probabilities of flooding. The literature review also revealed that there is not any maintenance strategy including flooding in the life-cycle. Although a few studies did flood risk assessment, they used nominal roads, and no after flood pavement performances were analysed.

In view of that, the current study developed the project and network levels roughness and rutting based RD models incorporating flooding and pre- and post-flood maintenance strategies. The RD models can provide road deterioration for short period up to 2 to 3 years after a certain probability of a flood, and help getting appropriate after flood treatments for rehabilitation. The pre- and post-flood strategies also used these models for after flood pavement performances prediction and an optimum treatment selection. The proposed strategies provide optimal solution in managing roads considering a flooding event in LCA.

The RD modelling and their verification results assisted using the two new indicators to determine pavement performances after a flood. These indicators, i.e., ΔIRI/Pr and ΔIRI/MrL, provided values on change in roughness due to a flood. Similarly, a flood risk assessment also gave sound results on consequences of pavement performances using IRI changes due to a flood. The lower the consequence or change in roughness, the better is the performance.

The above findings provide valuable outcomes in improving a PMS including flooding. Any road authority may use the RD models, pavement performances and resilience results' after a flood, the pre-and post-flood strategies right now; or may consider the current approaches for obtaining the same. It will ensure an efficient and cost-effective life cycle asset management with flooding. The research also suggests converting the flood damaged roads into stronger ones. As a result, a pavement would perform better in the life-cycle at the reduced maintenance costs.

11.2 Major Findings to Address the Objectives

The research identified gaps and set objectives accordingly, which was discussed in Chapter One and in the previous section. Section 2.10 reveals literature review and current gaps. Detailed approaches for achieving the objectives are shown in Chapter Three. This section provides key results on the RD models with flooding, pavement performances due to Mr loss, pavements' flood resilience, optimum M&R, pre- and post-flood strategies and flood risk assessment through pavement performances.

11.2.1 Project and Network level RD models

The study considered percentage transition method (probabilistic approach) for developing non-homogeneous TPMs using the roughness and rutting vs. time data. These TPMs were based on with and without flooding, and were used in the Monte Carlo simulation for getting the roughness and rutting based RD models at different probabilities of flooding. The RD models with flooding were generated for representative road groups and some selected roads. They were validated with the actual data and t-tests. The models provide pavement performances after a flood for short period up to 2 to 3 years.

Chapter Four shows the detailed RD modelling results. As an example, the RD modelling results of two specific road groups were shown. As expected, the generated RD models with higher probabilities of flooding showed an increase rate of deterioration. It was observed that

ΔIRI and ΔRutting were high in the initial years due to flooding events, and increase with an increase in flooding probabilities. Moreover, it revealed that a stronger road performs better during flooding. Each RD model is valid for its' representative roads.

The research derived the site-specific RD models for four selected roads of Logan area, and compared the results with the network level RD models and actual data. It was found that the January 2011 flood was comparable to a 20% probability of flooding. The derived representative RD models at 20% probability of flooding were used to determine month-wise to up to two years deterioration for the effected roads, as these models were valid to these roads. An approach was also used to get ΔIRIf20% vs. time (month) and ΔRuttingf20% vs. time (month) for these roads, by deducting ΔIRI and ΔRutting at 20% and 0% flooding probability at a certain time. These provided month-wise pavement deterioration of these roads for the January 2011 flooding, which was found consistent to the actual data. In addition, the site-specific roughness-based RD model was compared with the network level RD model and actual data, and found reasonable matches. The results were also compared using a t-test.

Therefore, it indicates that the RD models with flooding can be used to predict pavements deterioration and ensure cost-effective road asset preservation after a flood. The following section also shows validation of these models.

11.2.2 Pavement performances due to Mr loss

As mentioned in *Figure 3.1*, the AASHTO and HDM-4 roughness models' were utilised to get pavement performances due to moisture intrusion and Mr losses at granular and subgrade layers. The analysis assumed that moisture intrusion effects on the untreated layers only and traffic loading was the same before and after a flood. *Table 3.1* showed the key calibration factors of these models. The derived Mr losses due to a change in ($S-S_{opt}$) were found consistent with the previous studies. Loss in Mr values at the granular and subgrade layers due to saturation/flooding were utilized as the key inputs for the AASHTO 2008 and HDM-4 models for predicting ΔIRI in year 1.

Detailed results are shown in Chapter Five. It was found that both the HDM-4 and AASHTO 2008 results could match well if appropriate SN values were used. It is worth noting that the

HDM-4 results were sensitive to the initial SN values and different layer thicknesses. In general, at the same loading level, the poor level pavement performs the poorest, followed by the fair one. Moreover, at the same strength level, the lower traffic loading road group performs the poorest, then the medium traffic loading road group with the high traffic road group performs the best. It shows that if the standards are the same for different loading roads, then the low traffic loading road groups perform the best as anticipated. In all the cases, exponential increases in roughness values were observed beyond the 30% loss in subgrade Mr and 22% Mr loss in granular layers. Moreover, ΔIRI was always found higher for the HDM-4 cases after this critical value.

The predicted ΔIRI in year 1 using the AASHTO 2008 and HDM-4 were found consistent to the actual after flood data, which was verified using two major roads data of Logan for the January 2011 flood. A t-test also supported the results. These outcomes were also consistent with the RD modelling results.

The study has observed some interesting findings, i.e., i) change in ($S-S_{opt}$) could vary between 45 and 75% after a flooding event; ii) the best flood-resistant pavement could be a Strong and High Standard road among the flexible pavement road groups, i.e., the F_HT_S, then F_MT_S and F_HT_F, as they had lower deterioration rates derived from the ΔIRI/MrL; and iii) a road (in this case F_HT_P) performed poorer in life-cycle if the Mr loss in year 1 due to moisture intrusion was higher, and it needed an additional thin overlay to perform at the set standard.

This pavement performance analysis due to granular and subgrade Mr loss had the following major limitations:

- No geotechnical investigations were possible during or immediately after a flood.
- Material non-linearity was not considered in the MET design due to not having any data during extreme moisture intrusion/a flood. This was also not in the scope.
- ($S-S_{opt}$) and Mr loss values were obtained using the EICM climate model of the AASHTO 2008 only, because of non-availability of actual Mr loss data.

- Validation of the predicted ΔIRI in year 1 was done using only two major roads' data in Logan of Queensland. No other roughness data after a flood and probability of flooding were available to verify the predicted results.

However, these findings gave a verification of the proposed RD models with flooding. All of the above findings were used to estimate pavement's flood resilience.

11.2.3 Pavements' flood resilience

Although several studies have observed pavement responses with flooding, no quantification on different pavement performances were undertaken to obtain a robust answer. The current study estimated performances of different pavement types with flooding, which helped in identifying sound pavements. Results obtained from the two new gradients, that is, ΔIRI/Pr and ΔIRI/MrL, and different types of pavements performances with flooding provided valuable information to obtain a flood-resilient pavement. It was observed that a strong, rigid and high standard road could perform the best with flooding.

A flood damaged road may be upgraded into a flood-resilient one as a proactive approach of asset preservation. A pavement's strength may be enhanced through strengthening overlay and/or layer stabilisation. Moreover, a road may be converted into a rigid or composite pavement through granular and subgrade layer stabilisation. The outcome ensured better pavement performances, and reduced service life maintenance costs with flooding. It was not possible to obtain results for all the road groups due to inconsistent data for some of the groups in RD modelling. The Mr loss analysis was only done for some flexible pavement road groups. However, these results were adequate to have confidence in obtaining pavements' flood resilience.

The study also derived the optimum M&R, pre- and post-flood strategies, which approaches are shown in Chapter Three. The following sections provide the key results.

11.2.4 Optimum strategy at a without flood scenario

Detailed findings on an optimum life cycle strategy at without flood condition were discussed in Chapter Seven. The well-known HDM-4 model was used to obtain the optimum standards and strategies of TMR-QLD with the optimisation objective of “minimising agency cost and maximising performance”. The current approach used an economic optimisation to get the best solution for a road group.

This analysis revealed results for all the flexible and composite pavement road groups. For example, results were provided for a common road group, i.e., F_MT_P. A maintenance strategy provides information on when to intervene and also recommends necessary treatments, standards and budget for a network. For any pavement type and traffic loading, the strong road groups performed better with higher standards (lower IRI) and the poor road groups performed poorer. Considering varying loading, a high loading road needed a better standard to manage it efficiently. Again, at the same standard, the high loading roads needed higher budget than the low or medium loading roads.

The results showed that the flexible and composite pavement road groups were sensitive to strength and loading, respectively. The composite roads had higher standards in comparison to the flexible roads, and a flexible pavement group needed more budget than a composite one. Although higher budget was needed to maintain a road at a better standard, it was not always true as higher budget was required for the poor and high loading roads to keep them at set standards.

About $17.8bn was needed to maintain the flexible and composite roads at 4.0 IRI in the next 20 years. That equates to $0.9bn per year or $0.04m/km in each year. The high loading roads were maintained at 3.0 IRI and the medium loading roads at 4.0 to 4.5 IRI. For example, the F_MT_P road group had to be maintained at 5.0 IRI with OV 45 mm which delivered the highest NPV/Cost of 1.25; NPV of $3,853m and IRR of 16.6%. This 4,344 km road group needed RM + OV 45 mm in its life-cycle, where OV 45 mm had to be provided in 2019 (year 6). It required $4,178.8m budget in the next 20 years, which was $208.9m per year and $0.048m/yr/km. Moreover, a budget optimisation indicated that the F_MT_P needed 100% of budget to keep it at 5.0 IRI.

Compared with the previous CRC CI study, the current results proposed higher standards to ensure better pavement performances. Both sets of analyses provided similar budget requirements for the matched road groups. The previous study did not provide an optimum strategy, and allowed roads to deteriorate completely; it recommended several treatments (higher agency costs) with poor pavement performances. It was shown that the proposed optimal strategy kept the roads at higher standards. All these would strengthen the current PMS and help the TMR-QLD manage its 34,000 km road asset efficiently. A road authority can use the current approach to develop optimum maintenance standards and strategies for efficient asset preservation.

11.2.5 Post-flood road maintenance strategy

A new approach to the development of a post-flood maintenance strategy was proposed in Chapter Eight. In the past, normal deterioration was used to select a post-flood rehabilitation, which may not be necessarily correct. This research used the newly derived roughness and rutting based RD models to predict deterioration in year 2, 3 or 4 assuming a flood in year 1. As a result, it was possible to obtain appropriate treatments. The thesis used the 34,000 km road database of the State road network in Queensland as a case study to derive the RD models with flooding and post-flood strategy.

The HDM-4 model was used to obtain a post-flood strategy. Three optimisation objectives were considered, i.e., i) maximise NPV, ii) maximise ΔIRI and iii) minimise agency cost at target IRI; which provided three different types of post-flood strategies. In addition, a 67% constrained budget optimisation revealed another set of results. All these results were compared considering agency cost, budget, economic benefits, pavement performances and suggested treatments. The analysis ultimately recommended two solutions, i.e., the constrained and unconstrained budget strategy. The constrained budget strategy using the 'minimise agency cost at target IRI' optimisation objective mostly suggested for thinner overlay at varying years, and could maintain the network at a set target. Therefore, it is an optimum solution subject to budget and performance constraints. It required $26.1bn in life-cycle if one flood was used, whereas $17.8bn was needed for no flood condition. This extra $8.3bn covered 23,660 km of the representative flexible and composite roads, which was close to the TMR-QLD's TNRP program. A road authority has to properly investigate its flood damaged roads before any implementation. On the other hand, if budget was unconstrained, then the results obtained with the 'maximise ΔIRI' could be chosen, which could keep the average road network at 1.5 to 2.0 IRI in its life-cycle.

Chapter Eight assumed a flood at year 1 in life-cycle. If this flood comes in any other year, then additional maintenance costs would be required at $0.9bn per year from year 1 to the before flood year. Moreover, if two or more floods occur, separate analyses are needed to predict after-flood road deterioration using the RD models at the specific probability of flooding. Finally, an HDM-4 analysis would give different post-flood strategy with extra costs. As an example, pavement performances with different post-flood strategies for a road group were shown. The importance of using the newly derived RD models with flooding to predict after-flood deterioration before any rehabilitation was also provided. This approach of road deterioration prediction with the RD models after a flood for at least 2 years before employing any rehabilitation is also valid for two or more floods in a life-cycle.

11.2.6 Pre-flood road maintenance strategy

A new approach in deriving a pre-flood road maintenance strategy was proposed in Chapter Nine. It intended to strengthen pavements structurally before a flood using a thin overlay as per the set optimum strategy, and then assessed pavement life-cycle performances if a flood comes at different years. An after flood treatment strategy was incorporated as a part of this pre-flood strategy.

In the analysis, a normal deterioration was assumed after a treatment given in year 1. However, the new roughness and rutting based RD models were used to predict after flood deterioration for at least two years before employing the post flood treatments. The study utilised HDM-4 for obtaining an after flood strategy. The results provided life-cycle pavement performances, necessary treatments at year 1 and any year after a flood, and required budget.

The treatment cost in year 1 as per the set strategy was about $21.13bn, which would enhance pavement structural strength now. As a result, the normal deterioration rate was slowed down and a thin overlay would require for after flood rehabilitation if a flood comes in any year, for example, year 2, 4, 7, 12 or 17. The pre-flood strategy considered a treatment in year 1 for a road group to increase pavement strength for better performances with flooding and another one after a flood. As a result, an after flood rehabilitation under a pre-flood strategy became cheaper with a thin overlay and could maintain the network better. The after flood treatment cost varied between $15.80bn and $17.00bn; totalling the pre-flood strategy costs around

\$37bn to \$38bn for a flood of 20 years span. Finally, the results revealed pavement life-cycle performances for the whole network and all road groups with treatment in year 1 and a flood in year 2, 4, 7, 12 or 17. The approach used here is valid even if two or more flood comes in the life-cycle. The average network life-cycle performances with the optimum, pre- and post-flood strategies were also shown.

All these pre- and post-flood strategy results were compared. Although the constrained post-flood strategy was cheaper at \$26.10bn; it was a reactive strategy and could not maintain the network better than any of the pre-flood strategies. Moreover, all the pre-flood strategies were economically beneficial and provided positive NPV, and others were not. Therefore, the derived pre-flood strategy is preferred and effective.

11.2.7 Flood risk assessment on pavements performances

The current research proposed a new approach for a flood risk assessment to evaluate pavement performances before and after a flood, and used the whole road network data of the Queensland’s main roads authority. Detailed results are given in Chapter Ten.

A flood for a road group was assumed if there was an abrupt increase in roughness among the IRI vs. time data. The probability of a flood was used for likelihood estimation; and roughness changes before and after a flood has been utilized to calculate consequence. It is worth noting that the immediate effect of a flood on pavement performances using IRI up to year 1 was used in the analysis, and no long term roughness deterioration trend was considered.

A detailed review was done on consequence scores for the roads who had the same probability of flooding. It appeared that a strong and high traffic loading road with high standard performed the best. However, the analysis revealed that mostly rigid pavements were at low to moderate risks, which cannot theoretically be supported and did not match the findings in obtaining flood-resilient pavements. Results from the two new indicators: ΔIRI/Pr and ΔIRI/MrL found that a strong, rigid, and high traffic loading road with high standard was the most flood-resilient. The study has validated flood consequences and risk scores using the data from two roads in Logan. These results would help in upgrading flood damaged roads into flood-resilient pavements, mainly by stabilising granular and subgrade layers and/or increasing asphalt concrete thickness to enhance pavement structural strength.

The key restrictions of this flood risk analysis were: i) rational assumption required for some road groups to estimate likelihood if the return period of floods were not found in the IRI vs. time data, ii) same flood frequency and similar initial road conditions were not available to properly compare results on pavement performances, and iii) limited data were available to use for validation of the flood risk consequences and scores.

11.3 Future Work

The overall research findings helped identifying some scope for future work. For example, an investigation could be undertaken with real after flood data to check the change in ΔIRI due to varying granular and subgrade Mr. It could also check causes of incremental change of roughness and rutting after flooding events. The results could be compared with the project and network level RD models with flooding, HDM-4 and AASHTO 2008 roughness prediction models. A relationship may be derived between Mr loss vs. time due to a flood (extreme moisture intrusion) to represent realistic results on pavement SN loss. Rutting and cracking models may be explored to get a better understanding of the impact on Mr loss during a flood. A series of experiments should be conducted to derive the assumed parameters in the HDM-4 and AASHTO models.

The flood resilience parameter may be used in pavement structural and rehabilitation design along with the design loading to choose the pavement type and strength. A road authority may consider changing their roads into flood-resilient pavements.

In future, the TMR-QLD may select some sections randomly, and monitor their performances with recommended treatments and standards. It is believed that a MCA using economic, political, social, and environmental factors could be utilised if economic results cannot produce a sound optimum standard.

A comprehensive flood risk assessment is needed in future, as different flooding probabilities have different consequences on road performances. It could be done by comparing pavement performances of road groups with the same flood frequency and similar initial conditions. An actual after flood roughness data would help obtaining better results for this purpose. Better prediction on the probability of flooding is desirable. In addition, two or more floods may be used for a road group to obtain valuable information on consequences and risk scores. Furthermore, probability and consequence of a flood should be estimated at a project level, not just the group (network) level. It is noted that a risk rating based road grouping derived from geographical location and exposure to a flood would be useful for future research work.

Furthermore, quantification of the economic impact on roads due to a flood may be considered.

11.4 Concluding Remarks

The research identified specific gaps on road deterioration prediction after a flood and as a consequence, an appropriate treatment selection. There is not any maintenance strategy that addresses flooding. Therefore, this research: i) proposed site-specific and network level novel roughness- and rutting-based RD models, ii) estimated pavement performances with flooding to get flood-resilience network, iii) provided pre- and post-flood strategies and an optimum M&R lifecycle strategy, and iv) did a flood risk assessment on pavement performances. All these addressed the objectives and help in the advancement of a PMS. It is expected that following this study, the derived RD models, results on flood-resilient pavements and pre- and post-flood strategies enable road authorities in achieving sustainable management of road assets.

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Appendices

Appendix 4A: The code/logic used in the Monte Carlo simulation for the RD modelling

```
clear

NM=[ .99 .01 0 0 0;
     .005 .687 .207 .101 0;
     0  0  0 1 .0;
     0  0   0 1  0;
     0  0   0  0  1];
FM=[ 0 1 0 0 0;
      0 .003 .606 .391 0;
      0  0  0 1 0;
      0  0   0 .168  .832;
      0  0   0  0  1];
TM0= [ 1 0 0 0 0;
      0 1 0 0 0;
      0  0  1 0 0;
      0  0   0 1  0;
      0  0   0  0  1];
InitState=[1 0 0 0 0];
State1=[1 0 0 0 0];
State2=[0 1 0 0 0];
State3=[0 0 1 0 0];
State4=[0 0 0 1 0];
State5=[0 0 0 0 1];
InitIRI=[1;
      2;
      3;
      4;
      5];
floodprob=0.05;
Sample_Size=10000;
Max_Years=20;
SimulatedState(Sample_Size,Max_Years)=0;
State(Max_Years)=0;

for ii=1:Sample_Size
    TM=TM0;
    CurrState=InitState;
    SimulatedState(ii,1)=InitState*InitIRI;
    for jj=2:Max_Years

        floodrnd= rand(1);
        if floodrnd>floodprob
            TM=NM;
        else
```

```
        TM=FM;
      end
      staternd = rand(1);
      expectedProb = CurrState*TM;
      if staternd < expectedProb(1)
        SimulatedState(ii,jj)=1;
        CurrState=State1;
      elseif staternd < expectedProb(1)+ expectedProb(2)
        SimulatedState(ii,jj)=2;
        CurrState=State2;
      elseif staternd < expectedProb(1)+expectedProb(2) ...
                    + expectedProb(3)
        SimulatedState(ii,jj)=3;
        CurrState=State3;
      elseif staternd < expectedProb(1)+expectedProb(2) ...
                  +expectedProb(3) + expectedProb(4)
        SimulatedState(ii,jj)=4;
        CurrState=State4;
      elseif staternd < expectedProb(1)+ expectedProb(2) ...
          +expectedProb(3)+ expectedProb(4) + expectedProb(5)
        SimulatedState(ii,jj)=5;
        CurrState=State5;
      end
    end

end

for jj=1:Max_Years
 State(jj)= mean(SimulatedState(:,jj));
end
plot (State)
```

Appendix 4B: The representative road groups

Road Groups	Pavement type	Year of construction	Year for road cond.	Last surfacing	Length (km)	Assumed Width (m)	Avg. Roughness (IRI, m/km)	Potholes (No./km)	Boken Edge (m^2/km)	Total Cracking (%)	Rutting (mm)	Depression (%)	Ravelling (%)	Pavement thickness (mm)	Surface thickness (mm)	Assumed SN	Year for traffic	Total MT	Total NMT	Total AADT	%CV	ESAL
F_LT_P	Flexible	1975	2011	2003	4,399.00	7.5	3.54	5.0	5.0	5.5	5.59	1.5	3.0	115	25	1.07	2011	250	0	250	21.24	403,170
F_LT_F	Flexible	1986	2011	2006	3,696.00	7.5	3.09	3.5	3.5	3.0	5.23	1.4	2.5	160	35	1.49	2011	262	0	262	21.52	399,671
F_LT_S	Flexible	1996	2011	2008	1,534.00	7.5	2.81	3.0	3.2	2.8	4.57	1.3	2.4	180	45	1.77	2011	230	0	230	22.59	374,996
F_MT_P	Flexible	1976	2011	2002	4,344.00	9.5	3.24	4.2	5.0	5.0	5.92	2.3	2.5	125	25	1.12	2011	2,103	0	2,103	19.46	3,196,156
F_MT_F	Flexible	1986	2011	2005	4,313.00	9.5	2.87	3.5	3.0	2.7	5.36	1.8	2.3	185	50	1.89	2011	2,314	0	2,314	19.67	3,540,634
F_MT_S	Flexible	1997	2011	2008	2,533.00	9.5	2.58	1.5	1.7	2.0	4.80	1.3	2.0	240	75	2.62	2011	2,054	0	2,054	20.63	3,577,461
F_HT_P	Flexible	1977	2011	2000	541.00	12.5	2.85	3.5	3.0	2.7	5.37	1.8	2.3	155	25	1.29	2011	9,878	0	9,878	15.45	14,510,000
F_HT_F	Flexible	1988	2011	2004	967.00	12.5	2.47	1.3	2.6	2.0	5.31	1.7	2.1	235	50	2.16	2011	9,864	0	9,864	16.43	15,413,000
F_HT_S	Flexible	2000	2011	2007	425.00	12.5	2.20	1.0	2.0	1.5	4.73	1.0	1.1	325	75	3.09	2011	8,996	0	8,996	16.60	14,202,000
C_LT_P	Composite	1984	2011	2001	3.00	7.5	2.89	2.5	2.7	3.8	4.40	1.0	1.0	135	25	1.18	2011	501	0	501	25.83	662,453
C_LT_F	Composite	1996	2011	2004	59.00	7.5	2.86	2.3	2.3	3.5	7.02	1.3	1.1	170	35	1.54	2011	513	0	513	19.64	500,583
C_LT_S	Composite	2006	2011	2008	93.30	7.5	2.40	1.3	1.6	2.7	4.31	1.0	1.0	215	45	1.96	2011	313	0	313	21.75	474,433
C_MT_P	Composite	1981	2011	2000	64.19	9.5	2.85	2.0	2.4	3.8	8.35	1.5	1.2	165	25	1.34	2011	3,854	0	3,854	24.72	3,449,291
C_MT_F	Composite	1996	2011	2003	100.00	9.5	2.63	1.4	1.8	3.1	5.01	1.1	1.1	195	50	1.94	2011	4,445	0	4,445	15.22	4,674,512
C_MT_S	Composite	2005	2011	2008	406.64	9.5	2.28	1.0	1.1	1.8	4.05	1.0	1.0	215	75	2.48	2011	2,405	0	2,405	21.30	3,524,934
C_HT_P	Composite	1981	2011	1998	21.00	12.5	2.34	1.5	2.0	3.3	5.75	1.2	1.2	195	25	1.51	2011	17,720	0	17,720	12.00	27,036,337
C_HT_F	Composite	1992	2011	2002	71.11	12.5	2.16	1.0	1.1	2.0	4.52	1.0	1.0	265	50	2.33	2011	15,188	0	15,188	13.79	27,742,320
C_HT_S	Composite	2003	2011	2007	90.59	12.5	1.69	0.6	0.5	1.0	3.63	0.7	0.8	340	75	3.17	2011	17,208	0	17,208	13.82	58,605,966
R_LT_P	Rigid	1952	2011	1998	2.11	7.5	2.81	2.3	2.0	3.0	6.10	1.3	1.2	90	25	0.93	2011	286	0	286	17.00	462,394
R_LT_F	Rigid	1976	2011	2004	9.17	7.5	2.97	2.0	1.8	3.5	6.40	1.4	1.1	110	35	1.21	2011	304	0	304	18.90	456,156
R_LT_S	Rigid	1987	2011	2007	15.85	7.5	2.93	1.5	1.6	2.5	5.60	1.2	1.2	165	45	1.69	2011	1,312	0	1,312	16.26	580,589
R_MT_P	Rigid	1964	2011	1990	2.63	9.5	3.79	3.3	1.6	6.8	8.20	1.8	1.3	125	25	1.12	2011	8,882	0	8,882	12.25	5,679,142
R_MT_F	Rigid	1987	2011	2000	12.13	9.5	2.83	2.0	1.3	3.1	5.60	1.2	1.1	145	50	1.67	2011	5,750	0	5,750	12.96	5,108,370
R_MT_S	Rigid	1993	2011	2006	11.00	9.5	2.58	1.5	1.1	2.9	4.70	1.1	1.0	200	75	2.40	2011	3,222	0	3,222	16.57	5,696,120
R_HT_P	Rigid	1963	2011	1971	2.06	12.5	4.93	4.0	2.5	8.3	10.30	2.5	1.8	185	25	1.45	2011	25,109	0	25,109	8.64	14,492,248
R_HT_F	Rigid	1988	2011	1996	12.00	12.5	2.35	1.1	1.3	2.0	4.50	1.1	1.0	215	50	2.05	2011	24,621	0	24,621	11.84	27,724,000
R_HT_S	Rigid	2001	2011	2003	17.00	12.5	1.74	0.6	0.5	1.1	3.65	0.8	0.8	345	75	3.20	2011	27,856	0	27,856	12.70	33,645,000

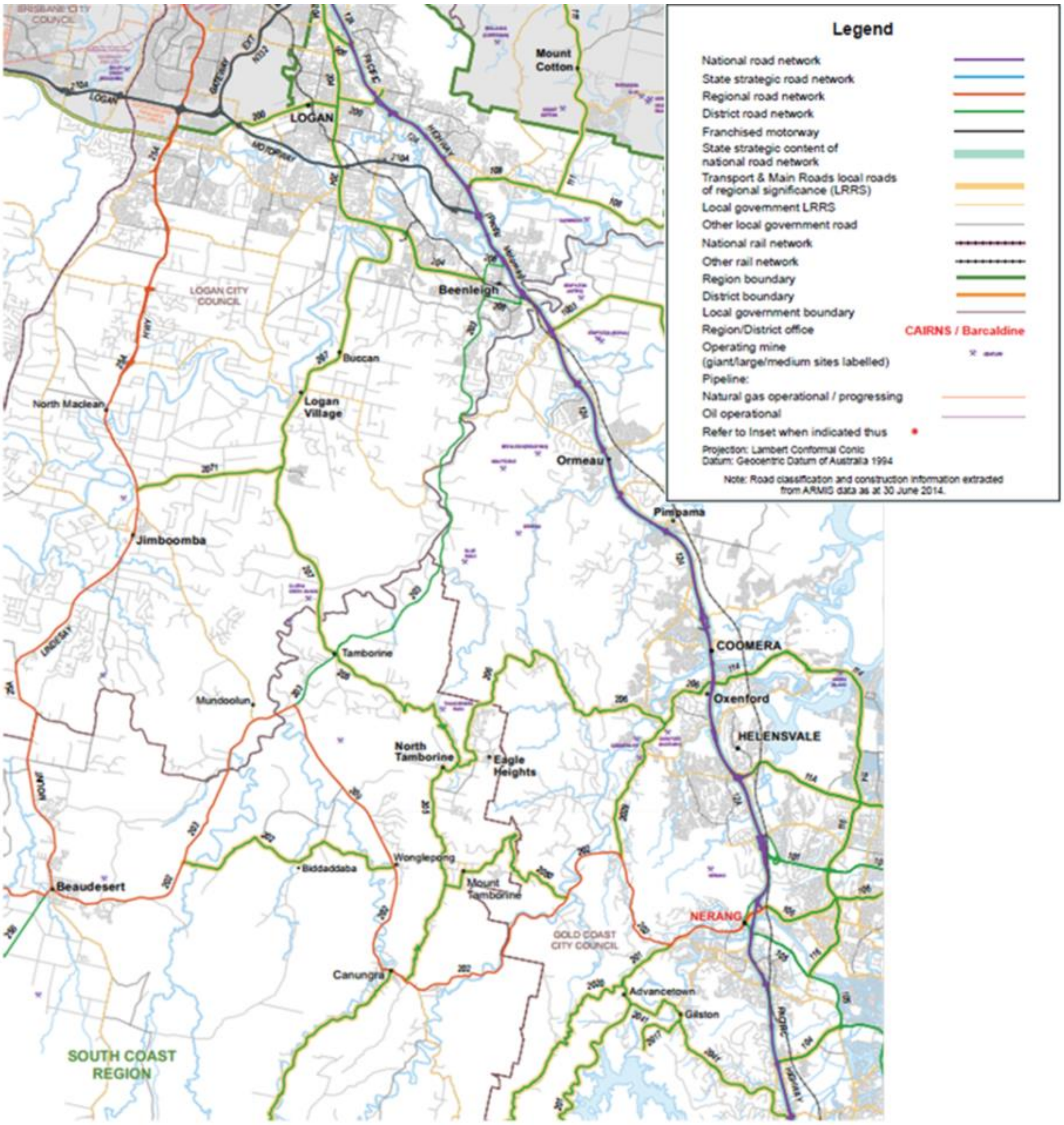
Appendix 4C: Comparison of the RD Modelling Results

Road Group	Set Optimum Standard	∆IRI Increase (in IRI)	∆Rutting Increase (in mm)	Final Remarks on Suitability
F_LT_P	5.5 IRI	0% prob: 0.62 50% prob: 0.73 100% prob: 0.85	0% prob: 0.92 50% prob: 2.22 100% prob: 3.53	IRI model range is small compared to the rutting one. IRI & Rutting-based RD models: OK ∆IRI/Rutting vs. time models: OK
F_LT_F	5.0 IRI	0% prob: 0.33 50% prob: 0.93 100% prob: 1.53		Performs better at low probability. IRI-based RD model: OK ∆IRI vs. time model: OK
F_LT_S	4.5 IRI	0% prob: 0.48 50% prob: 0.53 100% prob: 0.58	0% prob: 0.66 50% prob: 2.02 100% prob: 3.37	Considering the IRI model, a strong pavement performs well at the first year, and compared to a poor one. Considering the rutting model, a strong road performs better than a poor one. The results show better performance than HT; MT at lower probabilities. This road group performs better as it has a better standard. Therefore, the rutting-based RD model is a *good example*. IRI & Rutting-based RD models: OK ∆IRI/Rutting vs. time models: OK
F_MT_P	5.0 IRI	-	-	Wrong results.
F_MT_F	4.0 IRI	-	-	Wrong IRI model; as it provides the highest ∆ IRI at 0% probability of flooding or at normal condition.
F_MT_S	4.0 IRI	-	0% prob: 0.90 50% prob: 2.50 100% prob: 3.5	Considering the IRI model, this road group has higher standard than the F_LT_S, and hence it performs better. However, comparing with rigid and composite pavements, the F_MT_S should not perform better. Considering the rutting model, this road group performs slightly poorer than a low loading group. Although it has higher standard, higher loading effects on performance. Rutting-based RD model: OK ∆Rutting vs. time model: OK
F_HT_P	4.5 IRI	0% prob: 0.30 50% prob: 0.55 100% prob: 0.75	0% prob: 1.75 50% prob: 2.30 100% prob: 2.90	Considering the IRI model, this road group performs better than a F_LT_P; as it has higher standard. Considering the rutting model, the F_HT_P performs slightly better than a low loading road group at higher probability. IRI & Rutting-based RD models: OK ∆IRI/Rutting vs. time models: OK
F_HT_F	4.0 IRI	-	0% prob: 0.90 50% prob: 2.90 100% prob: 5.85	Wrong IRI model; as it provides the highest ∆ IRI at 0% probability of flooding or at normal condition. Considering the rutting model, the F_HT_F performs slightly better than a poor road group at low probability. Rutting-based RD model: OK ∆ Rutting vs. time model: OK
F_HT_S	4.0 IRI	0% prob: 0.35 50% prob: 0.48 100% prob: 0.62	-	Considering the IRI model, this road group performs better than a F_HT_P. The F_MT_S performs better than the F_HT_S due to low loading (both have the same standard). Again, this group performs slightly better than the F_LT_S because of higher standard. It also performs poorer than a rigid pavement group. A *good Example*. Considering the rutting model, this strong road group performs poorer than the fair and poor

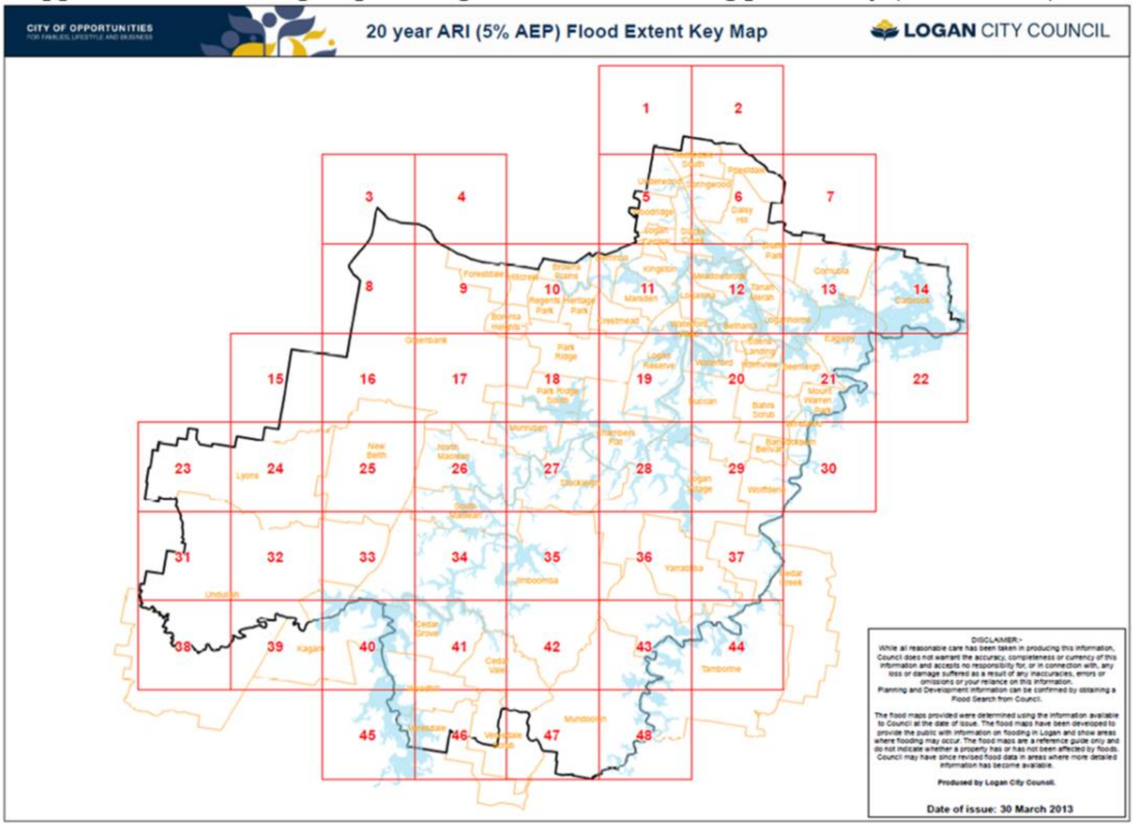
				strength road groups. IRI-based RD model: OK ΔIRI vs. time model: OK
C_LT_P	5.0 IRI	-	-	Wrong results.
C_LT_F	4.5 IRI	0% prob: 0.72 50% prob: 0.86 100% prob: 1.00	-	Considering the IRI model, this group performs better at high probability than a flexible pavement road group. Considering the rutting model, no Δ rutting was found at 0% probability of flooding or at normal condition. IRI-based RD model: OK ΔIRI vs. time model: OK
C_LT_S	3.5 IRI	-	0% prob: 2.63 50% prob: 3.13 100% prob: 3.67	Wrong IRI model; as it provides the highest Δ IRI at 0% probability of flooding or at normal condition. Considering the rutting model, this road group performs poorer than a F_LT_S. It is noted that a stabilised layer of a composite pavement becomes granular after some years; hence the composite and flexible pavements behave similar. The C_LT_S performs better than the C_LT_F. Rutting-based RD model: OK ΔRutting vs. time model: OK
C_MT_P	4.5 IRI	0% prob: 0.85 50% prob: 1.18 100% prob: 1.52	-	Not possible to compare with the flexible pavement and low loading road groups. IRI-based RD model: OK ΔIRI vs. time model: OK
C_MT_F	4.0 IRI	0% prob: 0.29 50% prob: 0.94 100% prob: 1.57	-	Considering the IRI model, this group performs better than the C_LT_F and C_MT_P at low probability. It has higher standard than the other two. IRI-based RD model: OK ΔIRI vs. time model: OK
C_MT_S	3.5 IRI	-	-	Considering the IRI model, no ΔIRI was found at 0% probability of flooding or at normal condition. Considering the rutting model, this group performs better than the F_MT_S and C_LT_S. Although the C_LT_S and C_MT_S have same standard, this group provides better performance which seems not reasonable.
C_HT_P	4.0 IRI	-	-	No data.
C_HT_F	4.0 IRI	-	0% prob: 2.91 50% prob: 3.76 100% prob: 4.63	Wrong IRI model; as it provides the highest ΔIRI at 0% probability of flooding or at normal condition. Considering the rutting model, this group performs better than the F_HT_F and C_LT_F at higher probability. It is worth noting that this road group has higher standard than the C_LT_F. Rutting-based RD model: OK ΔRutting vs. time model: OK
C_HT_S	4.0 IRI	0% prob: 0.43 50% prob: 0.70 100% prob: 1.00	-	Considering the IRI model, this group performs poorer than a flexible road. It is noted that a stabilised layer of a composite pavement becomes granular after some years, hence the composite and flexible pavements behave similar. It performs poorer than a C_MT_S as its standard is lower. Considering the rutting model, this group performs better than the F_HT_S, C_LT_S, C_MT_S and C_HT_F. Although the C_MT_S has better standard and lower loading, hence the C_MT_S has to perform better than the C_HT_S. Therefore, this rutting model is not chosen. IRI-based RD model: OK

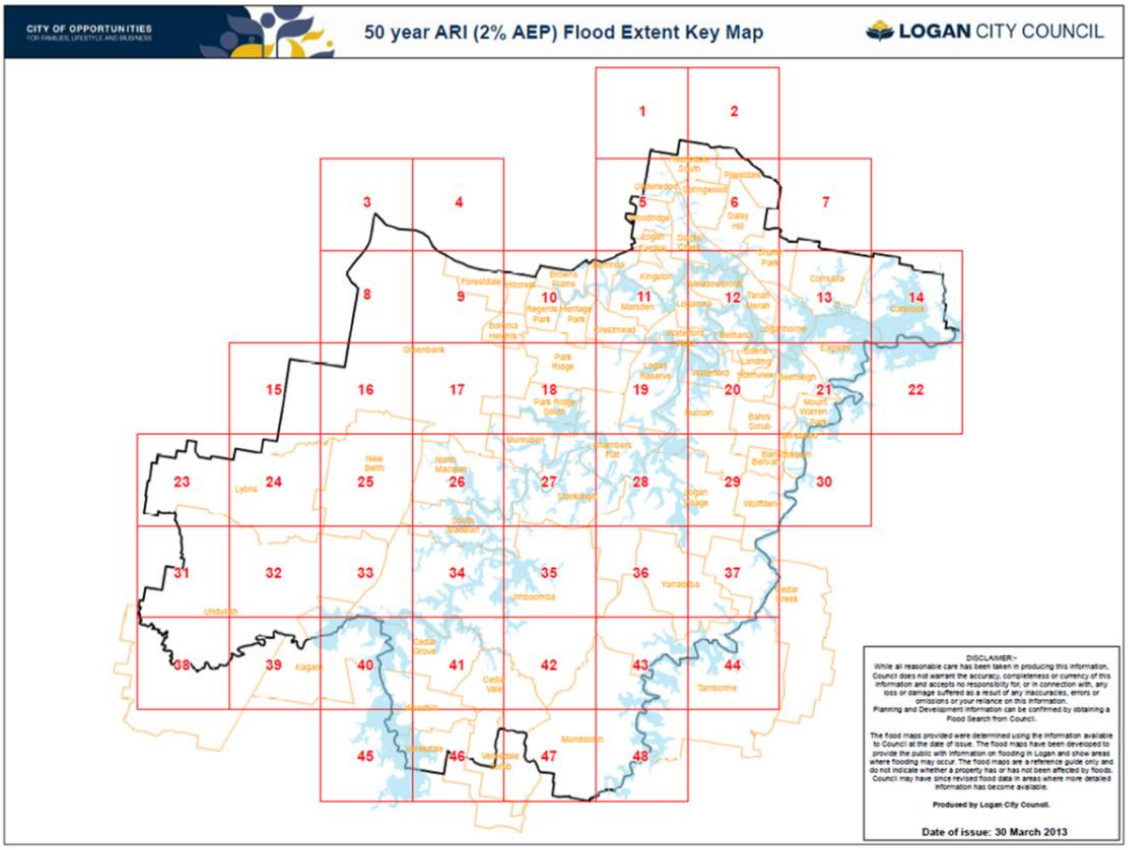
				ΔIRI vs. time model: OK
R_LT_P	Not available	-	No rutting model for a rigid pavement road group.	Wrong IRI model; as it provides no change at 0% probability of flooding or at normal condition. Again, this road group performs poorer than the F_LT_P at higher probability.
R_LT_F	Not available	0% prob: 0.01 50% prob: 0.51 100% prob: 1.00	No rutting model for a rigid pavement road group.	Considering the IRI model, this group performs better than the F_LT_F, C_LT_F and R_LT_P. It performs better than the moderate and high loading roads at lower probabilities. A *Good Example*. IRI-based RD model: OK ΔIRI vs. time model: OK
R_LT_S	Not available	-	No rutting model for a rigid pavement road group.	Wrong IRI model; as this road group performs poorer than a F_LT_S. Again, it performs worse compared to the poor and fair strength road groups.
R_MT_P	Not available	-	No rutting model for a rigid pavement road group.	Wrong IRI model; as it provides the highest ΔIRI at 0% probability of flooding or at normal condition.
R_MT_F	Not available	0% prob: 0.82 50% prob: 0.91 100% prob: 1.00	No rutting model for a rigid pavement road group.	Considering the IRI model, this group performs better than a C_MT_F at higher probability. It performs poorer that the R_LT_F and R_HT_F at lower probabilities. IRI-based RD model: OK ΔIRI vs. time model: OK
R_MT_S	Not available	-	No rutting model for a rigid pavement road group.	Considering the IRI model, this group performs poorer than the F_MT_S and C_MT_S. Again, this group performs poorer than a fair strength road group.
R_HT_P	Not available	-	No rutting model for a rigid pavement road group.	Wrong result.
R_HT_F	Not available	0% prob: 0.20 50% prob: 0.61 100% prob: 1.00	No rutting model for a rigid pavement road group.	Considering the IRI model, this road group performs poorer than a R_LT_F and better than a R_MT_F at low probability. IRI-based RD model: OK ΔIRI vs. time model: OK
R_HT_S	Not available	0% prob: 0.31 50% prob: 0.38 100% prob: 0.45	No rutting model for a rigid pavement road group.	Considering the IRI model, this group performs better than the flexible and composite roads. It performs better than the low and medium loading roads because of high standard as practised. A *good example*. IRI-based RD model: OK ΔIRI vs. time model: OK

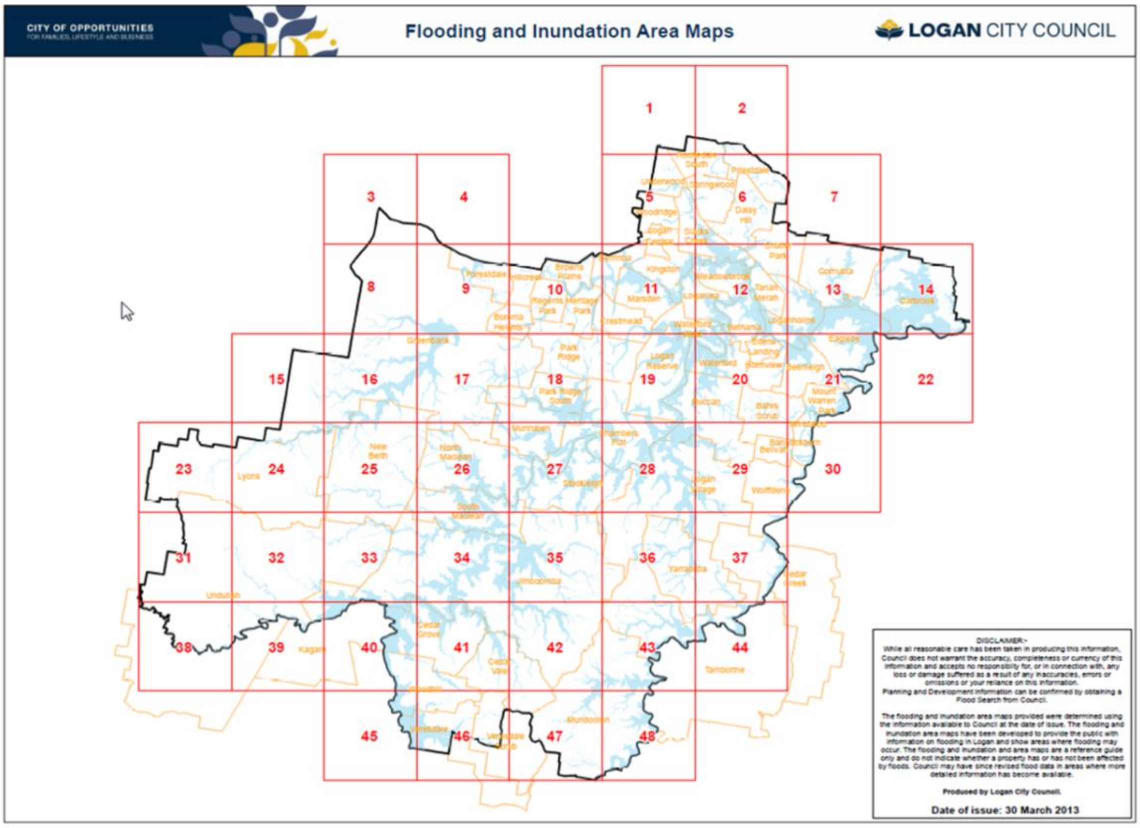
Appendix 4D: Major roads in the Logan city (TMR, 2014)



Appendix 4E: Flooding maps for Logan at the 20%, 50% and 100% flooding probability (LCC, 2015b)






CITY OF OPPORTUNITIES
FOR FAMILIES, LIFESTYLE AND BUSINESS
Flooding and Inundation Area Maps
LOGAN CITY COUNCIL
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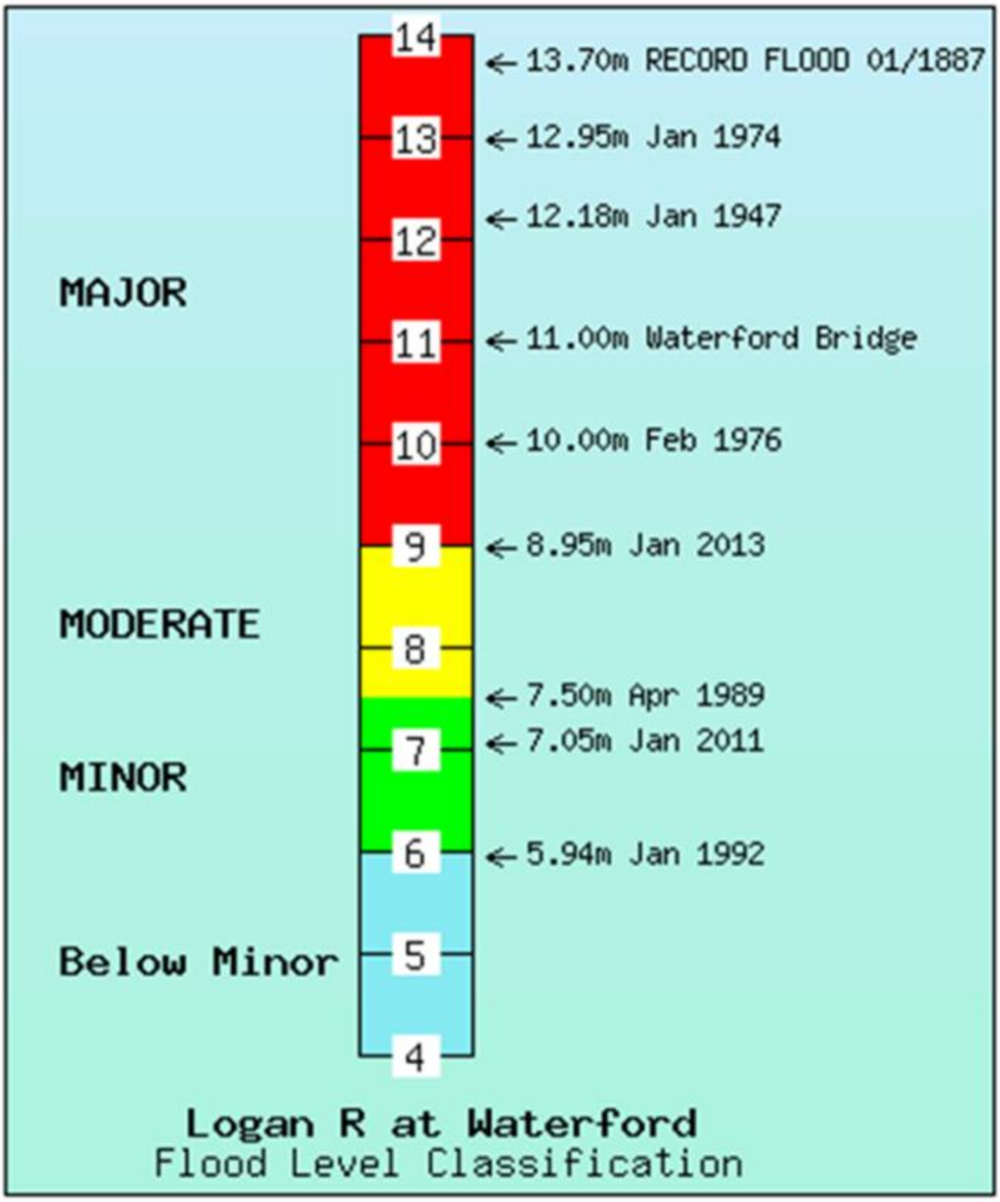
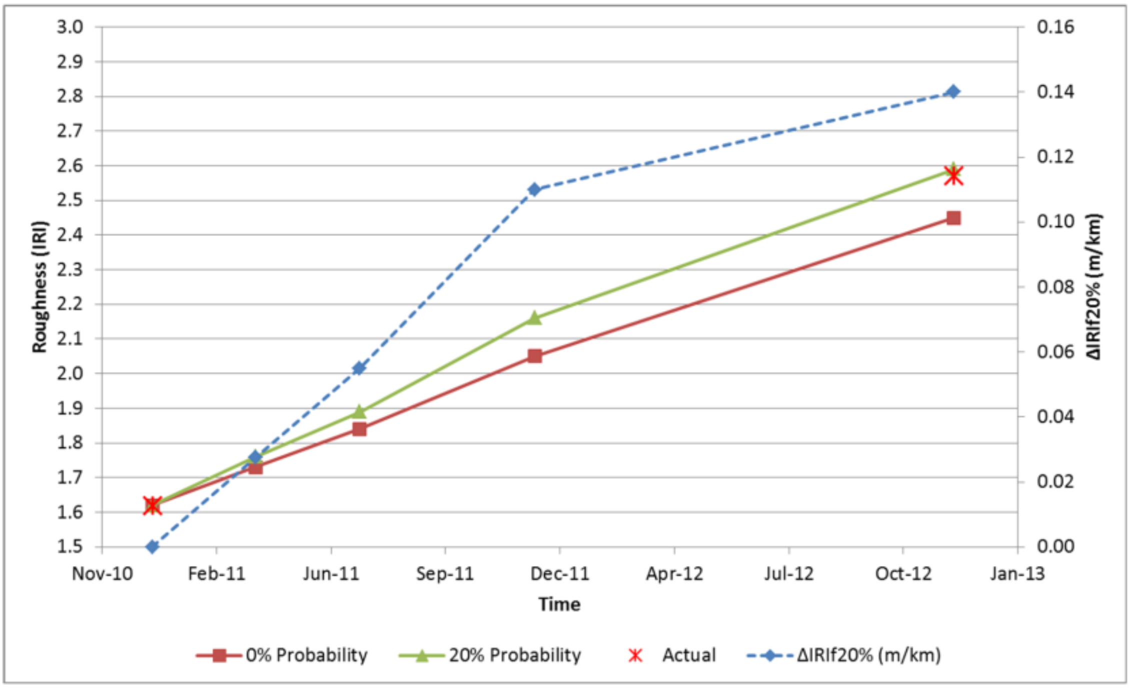
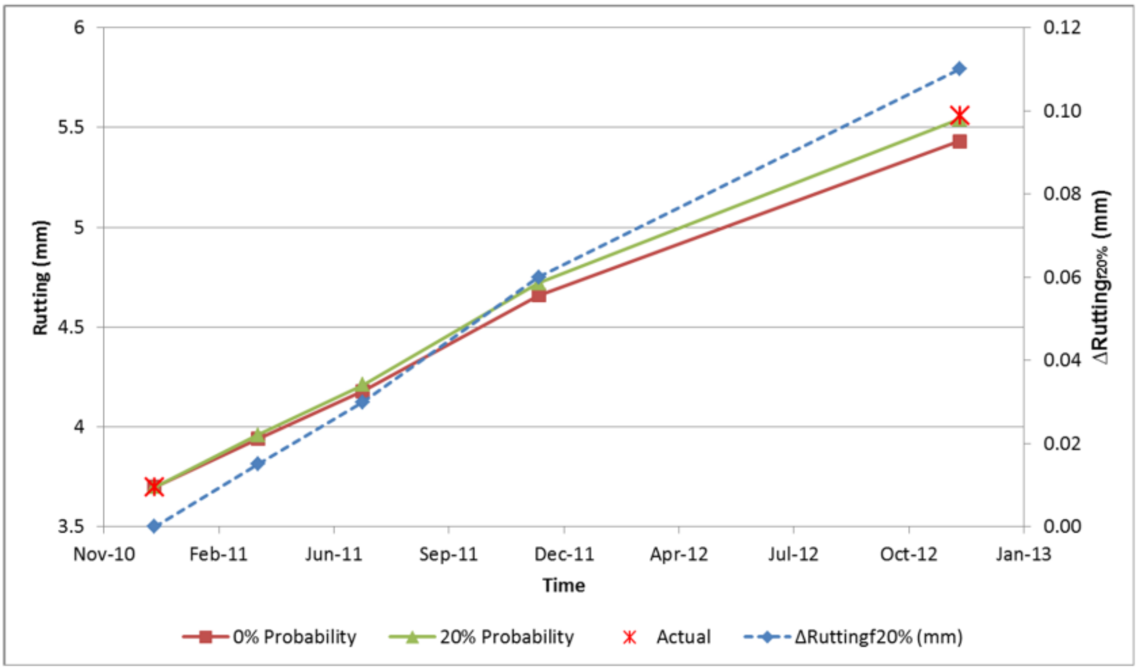
DISCLAIMER:-
While all reasonable care has been taken in producing this information, Council does not warrant the accuracy, completeness or currency of this information and accepts no responsibility for, or in connection with, any loss or damage suffered as a result of any inaccuracies, errors or omissions or your reliance on this information.
Planning and Development information can be confirmed by obtaining a Flood Search from Council.
The flooding and inundation area maps provided were determined using the information available to Council at the date of issue. The flooding and inundation area maps have been developed to provide the public with information on flooding in Logan and show areas where flooding may occur. The flooding and inundation and area maps are a reference guide only and do not indicate whether a property has or has not been affected by floods. Council may have since revised flood data in areas where more detailed information has become available.
Produced by Logan City Council.
Date of issue: 30 March 2013

Appendix 4F: Determination of flooding probability for the January 2011 flooding in Logan (BoM, 2014)

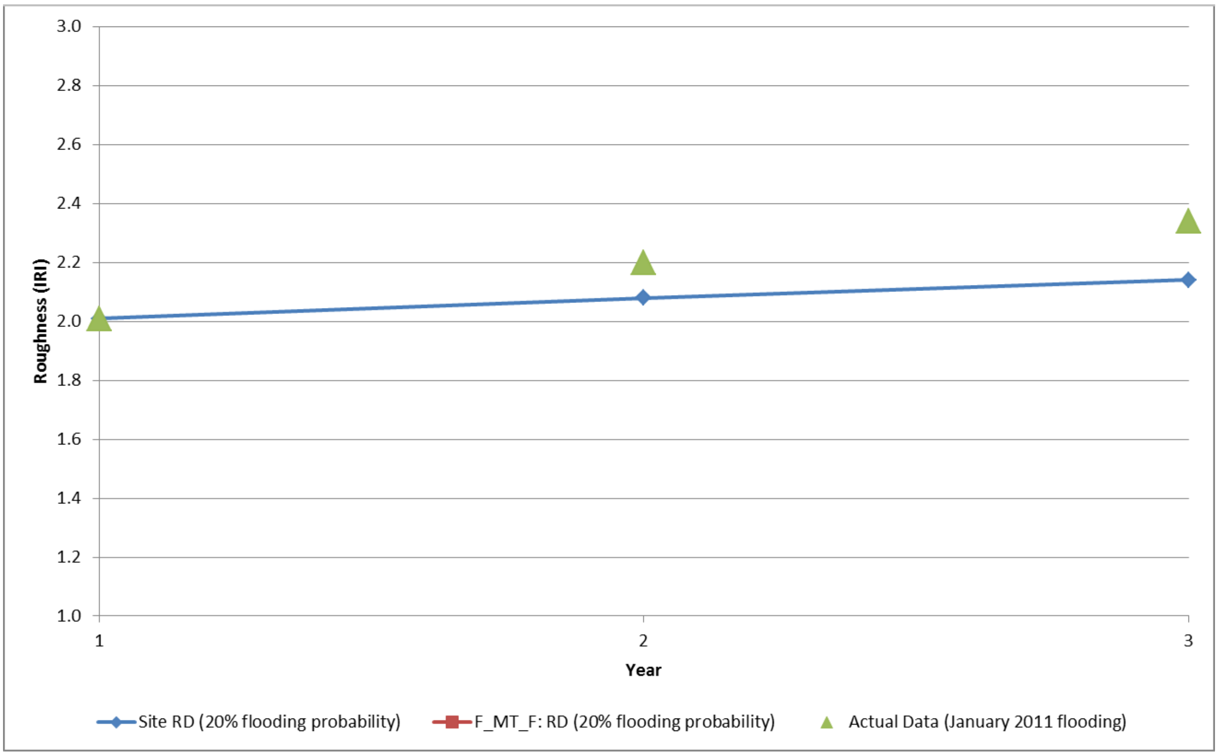


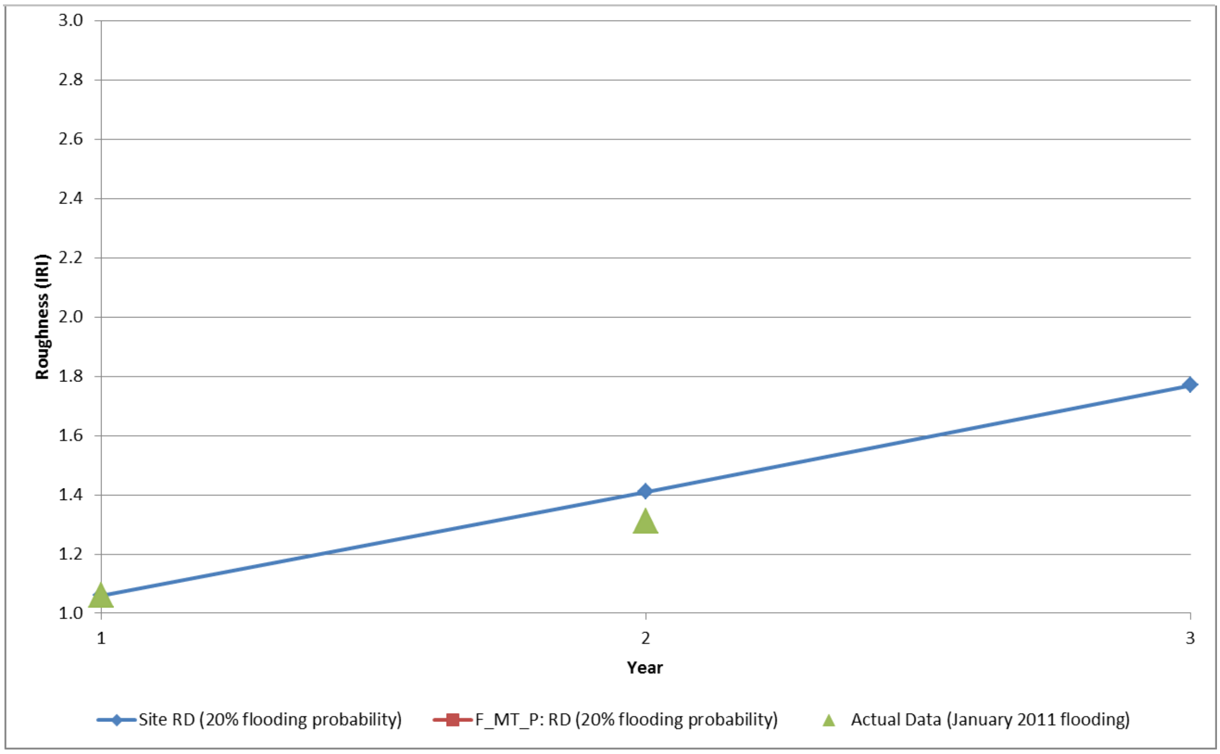
Appendix 4G: IRI vs time (month) and Rutting vs time (month) due to the January 2011 flooding, valid for the Beaudesert-Beenleigh Road

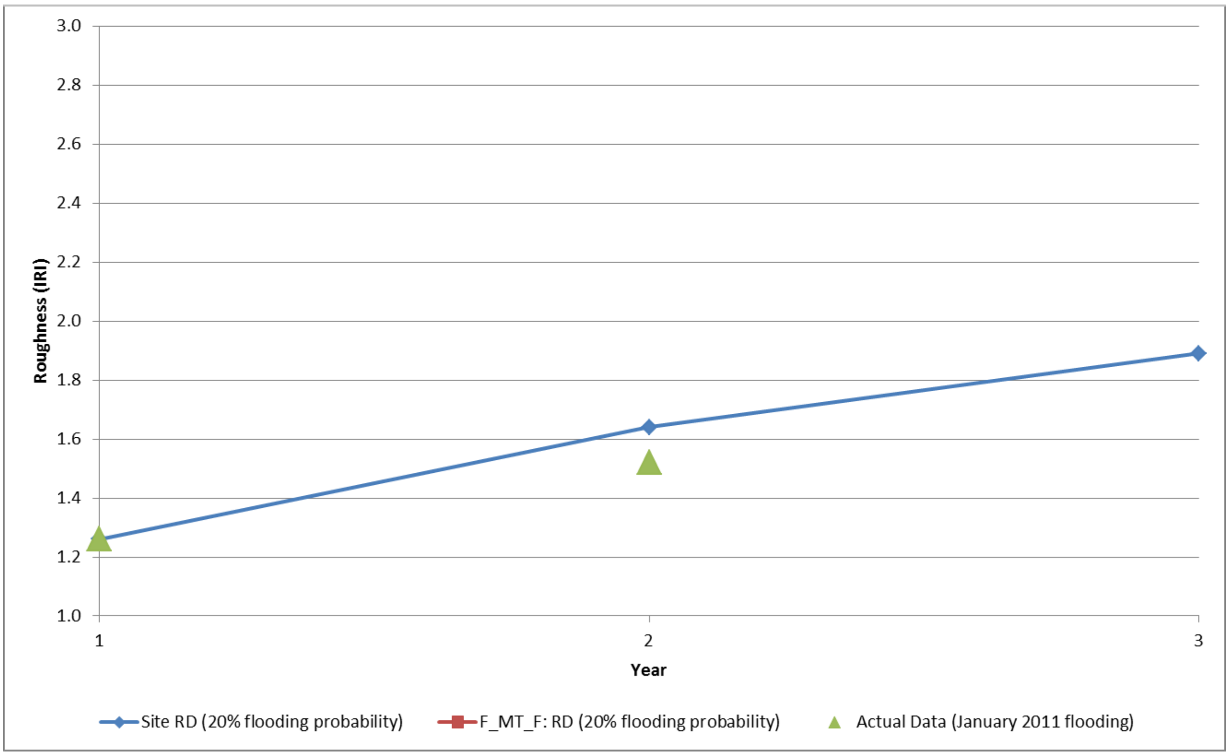




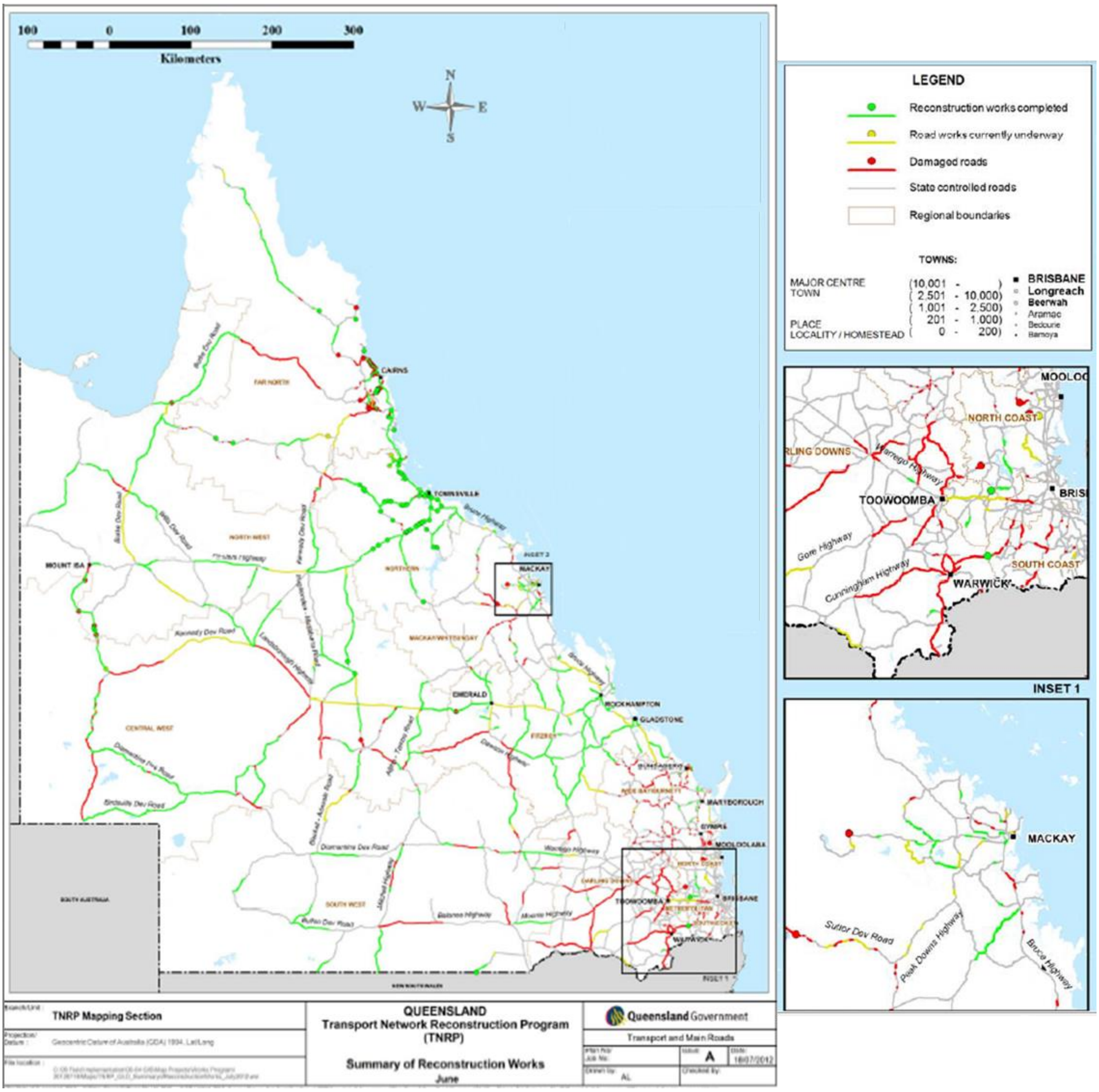
Appendix 4H: Comparison of the project level RD models with the actual data: a) F_MT_F part of the Waterford-Tamborine Road, b) F_MT_P part of the Beaudesert-Beenleigh Road, and c) F_MT_F part of the Beaudesert-Beenleigh Road






3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
Roughness (IRI)
1
2
3
Year
Site RD (20% flooding probability)
F_MT_F: RD (20% flooding probability)
Actual Data (January 2011 flooding)

Appendix 8A: The proposed TNRP after 2010/11 flooding



Appendix 8B: Post-flood maintenance strategy with 'maximise NPV'

Year	Road Group	Length (km)	Suggested Treatments	NPV/Cost	Financial Costs ($ m)
2013	C_HT_P	21.0	OV 200 mm at 3.0 IRI	0.937	61.688
	F_HT_P	541.0	OV 200 mm at 3.0 IRI	0.137	1,589.187
2017	C_MT_F	100.0	OV 30 mm at 4.0 IRI	0.822	71.250
	F_HT_F	967.0	OV 30 mm at 4.0 IRI	0.630	905.562
	C_MT_P	64.2	OV 30 mm at 4.0 IRI	0.485	45.735
	F_MT_F	4,313.0	OV 30 mm at 4.0 IRI	0.049	3,073.012
2018	C_HT_S	90.6	OV 30 mm at 4.0 IRI	6.011	84.928
	C_HT_F	71.1	OV 30 mm at 4.0 IRI	4.130	66.666
2019	F_HT_S	425.0	OV 30 mm at 4.0 IRI	1.534	398.438
2021	F_MT_P	4,344.0	Reconstruction at 12.0 IRI	0.000	2,421.606
2023	C_MT_S	406.6	OV 30 mm at 4.0 IRI	1.279	28.731
2024	C_MT_P	64.2	OV 30 mm at 4.0 IRI	0.485	45.735
2025	F_MT_S	2,533.0	OV 30 mm at 4.0 IRI	0.941	1,804.762
2026	F_MT_F	4,313.0	OV 30 mm at 4.0 IRI	0.049	3,073.012
2028	C_LT_P	3.0	Reconstruction at 12.0 IRI	0.000	1.320
2029	C_MT_F	100.0	OV 30 mm at 4.0 IRI	0.822	71.250
2030	F_HT_F	967.0	OV 30 mm at 4.0 IRI	0.630	906.562
	F_LT_P	4,399.0	Reconstruction at 12.0 IRI	0.000	1,936.000
	C_LT_F	59.0	Reconstruction at 12.0 IRI	0.000	25.966
Total					**16,873.413**

Appendix 8C: Post-flood maintenance strategy with 'minimise ∆IRI'

Year	Road Group	Length (km)	Suggested Treatments	NPV/Cost	Financial Costs ($ m)
2013	C_HT_P	21.0	OV 200 mm at 3.0 IRI	0.937	61.688
	C_HT_F	71.1	OV 200 mm at 3.0 IRI	0.505	208.886
	F_HT_P	541.0	OV 200 mm at 3.0 IRI	0.137	1,589.187
	C_MT_F	100.0	OV 200 mm at 3.0 IRI	-0.137	223.250
	C_MT_P	64.2	OV 200 mm at 3.0 IRI	-0.186	143.304
	F_HT_F	967.0	OV 200 mm at 3.0 IRI	-0.193	2,840.562
	F_MT_P	4,344.0	OV 200 mm at 3.0 IRI	-0.354	9,697.980
	F_MT_F	4,313.0	OV 200 mm at 3.0 IRI	-0.493	9,628.772
	C_MT_S	406.6	OV 200 mm at 3.0 IRI	-0.531	907.824
	F_MT_S	2,533.0	OV 200 mm at 3.0 IRI	-0.636	5,654.923
	C_LT_P	3.0	OV 200 mm at 3.0 IRI	-0.765	5.288
	C_LT_F	59.0	OV 200 mm at 3.0 IRI	-0.769	103.988
	F_LT_P	4,399.0	OV 200 mm at 3.0 IRI	-0.826	7,753.238
	F_LT_F	3,696.0	OV 200 mm at 3.0 IRI	-0.914	6.514.200
	F_LT_S	1,534.0	OV 200 mm at 3.0 IRI	-0.971	2,703.675
	C_LT_S	93.3	OV 200 mm at 3.0 IRI	-0.972	164.441
2015	F_HT_S	425.0	OV 200 mm at 3.0 IRI	-0.268	1,248.438
2017	C_HT_S	90.6	OV 200 mm at 3.0 IRI	1.331	266.108
Total					**49,715.751**

Appendix 8D: Post-flood maintenance strategy with 'minimise agency cost at target IRI'

Year	Road Group	Length (km)	Suggested Treatments	NPV/Cost	Financial Costs ($ m)
2013	C_HT_P	21.0	OV 200 mm at 3.0 IRI	0.937	61.688
	C_HT_F	71.1	OV 200 mm at 3.0 IRI	0.505	208.886
	C_MT_F	100.0	OV 200 mm at 3.0 IRI	-0.137	223.250
	C_MT_P	64.2	OV 200 mm at 3.0 IRI	-0.186	143.304
	C_MT_S	406.6	OV 200 mm at 3.0 IRI	-0.531	907.824
	C_LT_P	3.0	OV 200 mm at 3.0 IRI	-0.765	5.288
	C_LT_F	59.0	OV 200 mm at 3.0 IRI	-0.769	103.988
	C_LT_S	93.3	OV 200 mm at 3.0 IRI	-0.972	164.441
2017	C_HT_S	90.6	OV 200 mm at 3.0 IRI	1.331	266.108
	F_MT_P	4,344.0	OV 30 mm at 4.0 IRI	-0.209	3,095.100
	F_LT_P	4,399.0	OV 30 mm at 4.0 IRI	-0.620	2,474.437
	F_LT_F	3,696.0	OV 30 mm at 4.0 IRI	-0.836	2,079.000
2018	F_HT_P	541.0	Reconstruction at 12.0 IRI	0.000	396.823
	F_MT_P	4,344.0	OV 30 mm at 4.0 IRI	-0.209	3,095.100
2019	F_HT_F	967.0	Reconstruction at 12.0 IRI	0.000	709.295
2021	F_HT_S	425.0	Reconstruction at 12.0 IRI	0.000	311.738
2023	F_MT_F	4,313.0	Reconstruction at 12.0 IRI	0.000	2,404.325
2024	F_LT_P	4,399.0	OV 30 mm at 4.0 IRI	-0.620	2,474.437
	F_LT_F	3,696.0	OV 30 mm at 4.0 IRI	-0.836	2,079.000
2025	F_MT_S	2,533.0	OV 30 mm at 4.0 IRI	0.941	1,804.762
2026	F_MT_P	4,344.0	OV 30 mm at 4.0 IRI	-0.209	3,095.100
Total					**26,103.893**

Appendix 8E: Budget optimum post-flood maintenance strategy

Road Group	Length (km)	Normal Condition Optimum Standard	Suggested Treatment	Required Budget ($ m/yr/km)	Optimum Treatment as Post-flood Rehab	NPV ($ m)	NPV/Cost	IRR (%)	Budget Required for Post-flood Rehab ($ m)	Budget ($ m/yr/km)
C_HT_F	71.11	4.0 IRI	Overlay 30 mm	0.048	Overlay 30 mm @ 3.5 IRI in 2017	376.587	7.132	156.2	66.827	0.047
C_HT_P	21.00	4.0 IRI	Overlay 30 mm	0.048	Overlay 30 mm @ 5 IRI in 2017	117.701	7.548	731.5	20.13	0.048
C_HT_S	90.59	4.0 IRI	Overlay 30 mm	0.047	Overlay 45 mm @ 3 IRI in 2017	403.909	4.503	75	113.37	0.063
C_LT_F	59.00	4.5 IRI	Seal Coat 15 mm	0.024	Slurry Seal 10 mm @ 5.5 IRI in 2030	9.162	1.239	30.1	22.39	0.019
C_LT_P	3.00	5.0 IRI	Seal Coat 15 mm	0.013	Overlay 30 mm @ 5.5 IRI in 2028	0.151	0.214	25.8	1.797	0.030
C_LT_S	93.30	3.5 IRI	Overlay 30 mm	0.028	Overlay 30 mm @ 3.5 IRI in 2020	-32.224	-0.923	-18.8	53.78	0.029
C_MT_F	100.00	4.0 IRI	Overlay 30 mm	0.037	Overlay 30 mm @ 4.5 IRI in 2016	100.559	1.681	33.4	71.914	0.036
C_MT_P	64.19	4.5 IRI	Overlay 30 mm	0.037	Overlay 30 mm @ 5 IRI in 2017	43.908	1.212	44.3	46.654	0.036
C_MT_S	406.64	3.5 IRI	Overlay 30 mm	0.038	Overlay 30 mm @ 4.5 IRI in 2029	210.584	1.846	194.4	303.833	0.037
F_HT_F	967.00	4.0 IRI	Overlay 30 mm	0.047	Overlay 45 mm @ 4.5 IRI in 2017	3,377.732	3.528	88.3	1211.783	0.063
F_HT_P	541.00	4.5 IRI	Overlay 30 mm	0.093	Overlay 45 mm @ 5 IRI in 2016	1,943.155	3.422	104.6	686.628	0.063
F_HT_S	425.00	4.0 IRI	Overlay 30 mm	0.047	Overlay 30 mm @ 3.5 IRI in 2018	1,183.470	3.975	89.1	399.467	0.047
F_LT_F	3,696.00	5.0 IRI	Overlay 30 mm	0.029	Overlay 30 mm @ 5 IRI in 2016	-1,337.674	-0.766	-6.6	2107.209	0.029
F_LT_P	4,399.00	5.5 IRI	Overlay 30 mm	0.029	Overlay 30 mm @ 5.5 IRI in 2020	-558.131	-0.339	1.5	2547.238	0.029
F_LT_S	1,534.00	4.5 IRI	Overlay 30 mm	0.029	Overlay 30 mm @ 3.5 IRI in 2014	-744.633	-0.915	-12.4	863.183	0.028
F_MT_F	4,313.00	4.0 IRI	Overlay 30 mm	0.037	Overlay 30 mm @ 5 IRI in 2018	1,293.414	0.563	18.5	3144.276	0.036
F_MT_P	4,344.00	5.0 IRI	Overlay 45 mm	0.048	Overlay 45 mm @ 5.5 IRI in 2015	1,565.909	0.426	14.5	4157.766	0.048
F_MT_S	2,433.00	4.0 IRI	Overlay 30 mm	0.036	Seal Coat 15 mm @ 4.5 IRI in 2030 & 2031	1,021.957	1.962	59.7	1583.039	0.033

Appendix 9A: After flood strategy for the T1_F2_RD3&4_Post3

Road Group	Road Length (km)	Suggested Treatments (based on NPV/Cost)	Treatment Year	Costs ($ bn)	Budget ($ m/Yr/km)	Level of Service (LoS)
C_HT_F	71.11	Overlay 30 mm @ 4.0 IRI	2017	0.0667	0.0469	4.0
C_HT_P	21.00	Overlay 30 mm @ 4.5 IRI	2017	0.0197	0.0469	4.5
C_HT_S	90.59	Overlay 30 mm @ 4.0 IRI	2020	0.0851	0.0470	4.0
C_LT_F	59.00	Overlay 30 mm @ 4.5 IRI	2016	0.0332	0.0281	4.5
C_LT_P	3.00	Overlay 30 mm @ 5.0 IRI	2016	0.0017	0.0283	5.0
C_LT_S	93.30	Overlay 45 mm @ 3.0 IRI	2014	0.0699	0.0375	3.0
C_MT_F	100.00	Overlay 30 mm @ 4.5 IRI	2015	0.0713	0.0357	4.5
C_MT_P	64.19	Overlay 30 mm @ 5.0 IRI	2019	0.0458	0.0357	5.0
C_MT_S	406.64	Overlay 30 mm @ 4.5 IRI	2020	0.2899	0.0356	4.5
F_HT_F	967.00	Overlay 30 mm @ 4.0 IRI	2018	0.9080	0.0469	4.0
F_HT_P	541.00	Overlay 45 mm @ 5.0 IRI	2018	0.6770	0.0626	5.0
F_HT_S	425.00	Overlay 30 mm @ 4.0 IRI	2020	0.3990	0.0469	4.0
F_LT_F	3,696.00	Overlay 30 mm @ 5.0 IRI	2015	2.0796	0.0281	5.0
F_LT_P	4,399.00	Overlay 30 mm @ 3.5 IRI	2015	2.4752	0.0281	3.5
F_LT_S	1,534.00	Overlay 30 mm @ 4.5 IRI	2015	0.8630	0.0281	4.5
F_MT_F	4,313.00	Overlay 30 mm @ 5.5 IRI	2026	3.0743	0.0356	5.5
F_MT_P	4,344.00	Overlay 30 mm @ 5.0 IRI	2021	3.0969	0.0356	5.0
F_MT_S	2,533.00	Overlay 30 mm @ 3.5 IRI	2022	1.8055	0.0356	3.5
	Total: 23,660.83			**Total: 16.0618**		**Avg. network LoS: 5.1 IRI**

Appendix 9B: After flood strategy for the T1_F4_RD5&6_Post5

Road Group	Road Length (km)	Suggested Treatments (based on NPV/Cost)	Treatment Year	Costs ($ bn)	Budget ($ m/Yr/km)	Level of Service (LoS)
C_HT_F	71.11	Overlay 30 mm @ 4.5 IRI	2019	0.0667	0.0469	4.5
C_HT_P	21.00	Overlay 45 mm @ 5.0 IRI	2018	0.0263	0.0626	5.0
C_HT_S	90.59	Overlay 30 mm @ 4.0 IRI	2021	0.0851	0.0470	4.0
C_LT_F	59.00	Overlay 30 mm @ 4.0 IRI	2016	0.0332	0.0281	4.0
C_LT_P	3.00	Overlay 30 mm @ 4.0 IRI	2016	0.0017	0.0283	4.0
C_LT_S	93.30	Overlay 45 mm @ 3.0 IRI	2016	0.0699	0.0375	3.0
C_MT_F	100.00	Overlay 30 mm @ 5.5 IRI	2020	0.0713	0.0357	5.5
C_MT_P	64.19	Overlay 30 mm @ 5.5 IRI	2020	0.0458	0.0357	5.5
C_MT_S	406.64	Overlay 30 mm @ 3.5 IRI	2017	0.2899	0.0356	3.5
F_HT_F	967.00	Overlay 30 mm @ 4.0 IRI	2021	0.9080	0.0469	4.0
F_HT_P	541.00	Overlay 30 mm @ 5.0 IRI	2019	0.5071	0.0469	5.0
F_HT_S	425.00	Overlay 30 mm @ 4.0 IRI	2022	0.3990	0.0469	4.0
F_LT_F	3,696.00	Overlay 30 mm @ 4.0 IRI	2016	2.0796	0.0281	4.0
F_LT_P	4,399.00	Overlay 30 mm @ 4.0 IRI	2018	2.4752	0.0281	4.0
F_LT_S	1,534.00	Overlay 45 mm @ 3.0 IRI	2016	1.1507	0.0375	3.0
F_MT_F	4,313.00	Overlay 30 mm @ 5.5 IRI	2026	3.0743	0.0356	5.5
F_MT_P	4,344.00	Overlay 30 mm @ 5.5 IRI	2024	3.0969	0.0356	5.5
F_MT_S	2,533.00	Overlay 30 mm @ 3.5 IRI	2023	1.8055	0.0356	3.5
	Total: 23,660.83			**Total: 16.1862**		**Avg. network LoS: 4.9 IRI**

Appendix 9C: After flood strategy for the T1_F7_RD8&9_Post8

Road Group	Road Length (km)	Suggested Treatments (based on NPV/Cost)	Treatment Year	Costs ($ bn)	Budget ($ m/Yr/km)	Level of Service (LoS)
C_HT_F	71.11	Overlay 30 mm @ 5.0 IRI	2023	0.0667	0.0469	5.0
C_HT_P	21.00	Overlay 30 mm @ 4.0 IRI	2019	0.0197	0.0469	4.0
C_HT_S	90.59	Overlay 30 mm @ 4.0 IRI	2024	0.0851	0.0470	4.0
C_LT_F	59.00	Overlay 30 mm @ 4.0 IRI	2019	0.0332	0.0281	4.0
C_LT_P	3.00	Overlay 30 mm @ 4.5 IRI	2019	0.0017	0.0283	4.5
C_LT_S	93.30	Overlay 30 mm @ 3.0 IRI	2019	0.0525	0.0281	3.0
C_MT_F	100.00	Overlay 30 mm @ 5.5 IRI	2021	0.0713	0.0357	5.5
C_MT_P	64.19	Overlay 30 mm @ 5.5 IRI	2021	0.0458	0.0357	5.5
C_MT_S	406.64	Overlay 30 mm @ 3.0 IRI	2019	0.2899	0.0356	3.0
F_HT_F	967.00	Overlay 30 mm @ 4.0 IRI	2022	0.9080	0.0469	4.0
F_HT_P	541.00	Overlay 30 mm @ 5.0 IRI	2020	0.5071	0.0469	5.0
F_HT_S	425.00	Overlay 30 mm @ 4.0 IRI	2024	0.3990	0.0469	4.0
F_LT_F	3,696.00	Overlay 30 mm @ 4.0 IRI	2019	2.0796	0.0281	4.0
F_LT_P	4,399.00	Overlay 30 mm @ 4.0 IRI	2020	2.4752	0.0281	4.0
F_LT_S	1,534.00	Overlay 30 mm @ 3.0 IRI	2019	0.8630	0.0281	3.0
F_MT_F	4,313.00	Overlay 30 mm @ 4.0 IRI	2020	3.0743	0.0356	4.0
F_MT_P	4,344.00	Overlay 30 mm @ 4.5 IRI	2021	3.0969	0.0356	4.5
F_MT_S	2,533.00	Overlay 30 mm @ 3.0 IRI	2019	1.8055	0.0356	3.0
	Total: 23,660.83			**Total: 15.8745**		**Avg. network LoS: 4.7 IRI**

Appendix 9D: After flood strategy for the T1_F12_RD13&14_Post13

Road Group	Road Length (km)	Suggested Treatments (based on NPV/Cost)	Treatment Year	Costs ($ bn)	Budget ($ m/Yr/km)	Level of Service (LoS)
C_HT_F	71.11	Overlay 30 mm @ 4.0 IRI	2024	0.0667	0.0469	4.0
C_HT_P	21.00	Overlay 30 mm @ 4.5 IRI	2024	0.0197	0.0469	4.5
C_HT_S	90.59	Seal Coat 15 mm @ 5.5 IRI	2030	0.0351	0.0194	5.5
C_LT_F	59.00	Overlay 30 mm @ 4.5 IRI	2024	0.0332	0.0281	4.5
C_LT_P	3.00	Overlay 30 mm @ 5.0 IRI	2024	0.0017	0.0283	5.0
C_LT_S	93.30	Overlay 30 mm @ 3.0 IRI	2024	0.0525	0.0281	3.0
C_MT_F	100.00	Overlay 30 mm @ 4.0 IRI	2024	0.0713	0.0357	4.0
C_MT_P	64.19	Overlay 30 mm @ 5.0 IRI	2024	0.0458	0.0357	5.0
C_MT_S	406.64	Overlay 30 mm @ 3.0 IRI	2024	0.2899	0.0356	3.0
F_HT_F	967.00	Overlay 30 mm @ 4.0 IRI	2025	0.9067	0.0469	4.0
F_HT_P	541.00	Overlay 30 mm @ 4.5 IRI	2024	0.5071	0.0469	4.5
F_HT_S	425.00	Overlay 30 mm @ 3.0 IRI	2024	0.3990	0.0469	3.0
F_LT_F	3,696.00	Overlay 30 mm @ 4.5 IRI	2024	2.0796	0.0281	4.5
F_LT_P	4,399.00	Overlay 30 mm @ 3.5 IRI	2024	2.4752	0.0281	3.5
F_LT_S	1,534.00	Overlay 30 mm @ 3.0 IRI	2024	0.8630	0.0281	3.0
F_MT_F	4,313.00	Overlay 30 mm @ 4.0 IRI	2024	3.0736	0.0356	4.0
F_MT_P	4,344.00	Overlay 30 mm @ 4.5 IRI	2024	3.0969	0.0356	4.5
F_MT_S	2,533.00	Overlay 30 mm @ 3.0 IRI	2024	1.8055	0.0356	3.0
	Total: 23,660.83			**Total: 15.8225**		**Avg. network LoS: 4.4 IRI**

Appendix 9E: After flood strategy for the T1_F17_RD18_Post18

Road Group	Road Length (km)	Suggested Treatments (based on NPV/Cost)	Treatment Year	Costs ($ bn)	Budget ($ m/Yr/km)	Level of Service (LoS)
C_HT_F	71.11	Overlay 30 mm @ 4.5 IRI	2029	0.0667	0.0469	4.5
C_HT_P	21.00	Overlay 30 mm @ 5.0 IRI	2029	0.0197	0.0469	5.0
C_HT_S	90.59	Overlay 30 mm @ 3.5 IRI	2029	0.0849	0.0469	3.5
C_LT_F	59.00	Overlay 30 mm @ 4.5 IRI	2029	0.0332	0.0281	4.5
C_LT_P	3.00	Overlay 45 mm @ 5.0 IRI	2029	0.0023	0.0383	5.0
C_LT_S	93.30	Overlay 30 mm @ 3.5 IRI	2029	0.0525	0.0281	3.5
C_MT_F	100.00	Overlay 45 mm @ 4.5 IRI	2029	0.0950	0.0475	4.5
C_MT_P	64.19	Overlay 45 mm @ 5.0 IRI	2029	0.0610	0.0475	5.0
C_MT_S	406.64	Overlay 30 mm @ 3.5 IRI	2029	0.2899	0.0356	3.5
F_HT_F	967.00	Overlay 30 mm @ 3.5 IRI	2029	0.9067	0.0469	3.5
F_HT_P	541.00	Overlay 30 mm @ 5.0 IRI	2029	0.5071	0.0469	5.0
F_HT_S	425.00	Overlay 30 mm @ 3.0 IRI	2029	0.3990	0.0469	3.0
F_LT_F	3,696.00	Overlay 30 mm @ 4.0 IRI	2029	2.0796	0.0281	4.0
F_LT_P	4,399.00	Overlay 30 mm @ 4.0 IRI	2029	2.4752	0.0281	4.0
F_LT_S	1,534.00	Overlay 30 mm @ 3.5 IRI	2029	0.8630	0.0281	3.5
F_MT_F	4,313.00	Overlay 30 mm @ 4.0 IRI	2029	3.0736	0.0356	4.0
F_MT_P	4,344.00	Overlay 45 mm @ 5.0 IRI	2029	4.1274	0.0475	5.0
F_MT_S	2,533.00	Overlay 30 mm @ 3.0 IRI	2029	1.8049	0.0356	3.0
	Total: 23,660.83			**Total: 16.9417**		**Avg. network LoS: 4.25 IRI**