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ASSESSMENT AND MODELLING DETERIORATION OF FLOOD AFFECTED PAVEMENTS

A thesis Submitted in fulfilment of the requirements of the degree of

Doctor of Philosophy

by

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STATEMENT OF ORIGINALITY

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

Signed… ………Masuda Sultana

Date:..31/10/2016….

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DEDICATION

This dissertation is dedicated to my son Shayan and my husband Dolon, thank you for your love, patience, understanding and encouragement. You have given me unconditional support, cherished every great moment with me and supported me whenever I needed it.

LIST OF PUBLICATIONS AND AWARDS/ACHIEVEMENTS

During the course of this research work the following peer-reviewed papers have been published or submitted for publication.

JOURNAL PAPERS

1. Sultana, M., Chai, G., Martin, T. C., and Chowdhury, S. H. (2015). Modelling the Postflood Short term Behaviour of Flexible Pavements, *Journal of Transportation Engineering*, 142 (10) (Literature review is based on Chapter 2 and 3 and Analysis is included in Chapter 5).
2. Sultana, M., Chowdhury, S. H., Chai, G., and Martin, T. (2016). Modelling Rapid Deterioration of Flooded Pavements, ARRB Road and Transport Research Journal (in press-Analysis is included in Chapter 5),
3. Sultana, M., Chai, G. W., Chowdhury, S. H., and Martin, T. (2016). Deterioration of Flood Affected Queensland Roads - An Investigative Study, International Journal for Pavement Research and Technology (in press - Literature review section of this paper is based on Chapter 1 and analysis section is included in Chapters 5 and 6)
4. Sultana, M., Chai, G. W., Chowdhury, S. H., Martin, T., and Anissimov, Y. Modelling Rutting and Roughness and A State of the Art Review of Literature of Flood affected Pavements (based on Chapter 6 and 3 and will be submitted to Journal of Infrastructure Systems, ASCE in November, 2016)

CONFERENCE PAPERS

1. Sultana, M., Chai, G., Martin, T. Chowdhury, S. H. and Anissimov, Y. (2016), *A Statistical Analysis of Rapid Deterioration of Rutting and Roughness of Flood Affected Pavements in Queensland."* 27th ARRB Conference – Linking People, Places & Opportunities, Melbourne, November 16-18, 2016 (Analysis is included in Chapter 6).
2. Sultana, M., Chai, G. W., Chowdhury, S. H., and Martin, T. (2016). *Rapid deterioration of Pavements due to Flooding events in Australia*, Geo-China 2016, July 25-27, Shandong, China (Analysis is included in Chapter 5 and 6).
3. Sultana, M., Chai, G., Martin, T. and Chowdhury, S. H. (2015), *A Study on the Flood Affected Flexible Pavements of Australia*, 9th International Conference on

Road and Airfield Pavement Technology, August 9-13, 2015, Dalian, China (Analysis is included in Chapter 5).

4. Sultana, M., Chai, G., Martin, T. and Chowdhury, S. H. (2014), *A Review of the Structural Performance of Flooded Pavements,* 26th ARRB Conference – Research driving efficiency, October 19-22, 2014, Sydney, Australia (Literature review is based on Chapter 2 and 3 and Analysis is included in Chapter 5).

AWARDS/ACHIEVEMENTS

1. Awarded the Austroads fellowship for PhD study (scholarship for 3 years full time study) from Austroads and ARRB Group Ltd (formerly Australian Road Research Board) in 2013.
2. Awarded the National level (among Australian/New Zealand Universities) ARRB/Roads Australia Student Transport Research Prize 2015 for the Journal paper "Modelling Rapid Deterioration of Flooded Pavements" which was part of the PhD Research Project.
3. Awarded the Regional level (Queensland and Northern Territory) ARRB/Roads Australia Student Transport Research Prize 2015 for part of the PhD research project.
4. Conference Travel Grant from ARRB Group Ltd to attend the 27th ARRB Conference to be held in November, 2016.
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ABSTRACT

The efficiency of the road management system is increasingly challenged due to the frequent occurrences of extreme weather events, such as intense heavy rainfall, cyclones and flooding. The unpredictable events such as Hurricanes Katrina and Rita in the USA (2005), extreme flooding in January 2011 in South-East Queensland, Cyclones Oswald (2013) and Marcia (2015) in Queensland had severe impacts upon the road infrastructure. These roads are now subject to a wider range of climatic conditions over their service life than was originally anticipated during their design. To date, no deterioration model can accurately predict the impact of floods on pavements. To understand the impact of January 2011 flood on the structural performance of flood affected pavements, Falling Weight Deflectometer (FWD) deflection data and surface condition data (rutting and roughness), on flood-affected roads managed by Brisbane City Council, Department of Transport and Main Roads (TMR), Queensland and Roads and Maritime Services, New South Wales (RMS, NSW), Australia, were collected and examined.

The main aim of this research was to advance the knowledge and understanding of the impacts of extreme weather events, such as flooding, on pavement deterioration by assessing the damage caused by recent flooding events in Queensland and New South Wales, like January 2011 flood and 2013 floods. The other objective includes the development of mechanistic-empirical, deterministic-based pavement deterioration models for the prediction of rapid deterioration of structural and surface condition (rutting and roughness) of pavements impacted by river flooding or gradual increase in flood water.

Accurately predicting the post-flood short-term rapid deterioration phase of the pavement is the key to avoiding long-term consequences on road asset management. The pre- and post-flood data of the same pavement sections were used to assess the structural performance and surface condition of the partially or fully saturated pavements.

The original contribution of this research is the development of four mechanistic-empirical, deterministic-based deterioration models to accurately predict the rapid deterioration of structural and surface conditions (rutting and roughness) of flood affected pavements. All the four models can be used to predict the rapid deterioration phase of structural and surface conditions (rutting and roughness) of pavements following flood events. The models are sufficiently robust to be calibrated for the local conditions.

This research assessed the flood-affected pavements of Brisbane City Council, TMR, Queensland and RMS NSW. The assessment of the flood affected pavements indicated that these pavements deteriorated rapidly rather than gradually. This short-term phase was designated as rapid deterioration phase in this research. During the rapid deterioration phase, which starts immediately after the flood, accelerated reduction in structural condition and deterioration of surface condition occurred faster than expected during original design of pavements.

Apart from the key research contributions mentioned above, the study also delivered the following outcomes and made the following contributions:

- A structured methodology was established which can be used by road agencies to assess the impact of floods on pavements. This research provided evidence that pavements tend to fully or partially regain pre-flood structural strength after rehabilitation followed by dry weather periods in the area. The fact that some of the pavement sections investigated in this study did not fully regain their pre-flood strength is important as the road agencies will face increasing demand for rehabilitation in the future due to unanticipated deterioration.
- A comparison of observational data from Thin Asphalt Concrete (AC) pavements with gravel base and AC pavements with Cement Treated Base (CTB), demonstrated that latter category of pavements performed significantly better than the former category after the January 2011 flood in the Brisbane City Council area. Hence, AC pavements with CTB are recommended for future pavement construction for building road resilience in flood-prone areas.
- Analysis of data from TMR, Queensland indicated that pavements with low pre-flood rutting had significantly low post-flood rutting and pavements with high pre-flood rutting had significantly high post-flood rutting. These results suggest that pavements with low pre-flood rutting are highly likely to survive well after flooding while the opposite is true for pavements with high pre-flood rutting.
- This thesis also presents a general overview and strategic planning guidelines for investigation and long-term monitoring of flood affected roads which would improve the decision making process following any extreme weather event. The long-term monitoring of flood affected roads should be included in the decision making process of the road agencies that are at risk of frequent flooding.

The innovation of this thesis is the development of the models for flood-affected pavements. After calibrating for the local conditions, these models can be used in the pavement management system (PMS) of the local road agencies to predict deterioration of flood affected pavements which was not done by any other study previously. Moreover, the methodology followed in this study can be replicated by local road agencies in other countries that have flooding issues, to predict the deterioration of flood-affected pavements and to improve the efficiency of the PMS. The outcomes of the study will have direct and practical application in providing quantifiable engineering knowledge for the management of flood-affected roads.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHO	=	American Association of State Highway Officials
AASHTO	=	American Association of State Highway and Transportation Officials
AADT	=	Annual Average Daily Traffic
ARRB	=	Australian Road Research Board
BCC	=	Brisbane City Council
CBR	=	California Bearing Ratio
DCP	=	Dynamic Cone Penetrometer
ESAL	=	Equivalent Single Axle Load
FWD	=	Falling Weight Deflectometer
IRI	=	International Roughness Index
LTPP	=	Long-Term Pavement Performance
LTPPM	=	Long-Term Pavement Performance Maintenance
LTRC	=	Louisiana Transportation Research Centre
LVDT	=	Linear Voltage Differential transducers
MESA	=	Millions of Equivalent Standard Axles
NAASRA	=	National Association of Australian State Road Authorities
NDT	=	Non-Destructive Testing
PMS	=	Pavement Management System
PSI	=	Present Serviceability Index
SHRP	=	Strategic Highway Research Program
SN	=	Structural Number
SNC	=	Modified Structural Number
TMR	=	Transport and Main Roads, Queensland

1. INTRODUCTION

1.1 IMPACT OF EXTREME WEATHER EVENTS

Hurricanes Katrina and Rita devastated New Orleans and southeastern and southwestern portions of Louisiana, USA in 2005, and significantly damaged infrastructure (Gaspard et al. 2007). In recent years, Australia has also faced many extreme weather events such as frequent intense heavy rainfall events, flooding and droughts. Australia is a hot, dry country and has long been a land of weather extremes. The climate of Australia can be described as diverse because of its size and geographical location. It experiences extreme variable climate events ranging from droughts to floods (Chai et al. 2014). It covers a spectrum of climate classifications ranging from pure tropical over the northern quarter to alpine over the highlands of southern New South Wales, Victoria and Tasmania. The climate in Queensland ranges from subtropical in the south to tropical and equatorial rainforest climate in the north (Chai et al. 2014).

Climate change has been recognised globally as an issue of utmost concern and the threat of climate change poses problems to all nations (Garnaut 2011). Professor Garnaut's report on climate change serves as a timely reminder that the world is warming which is causing more droughts, water shortages and extreme weather events (Garnaut 2011). The report also expressed the concern that Australia is particularly vulnerable to climate change. Global climate has been changing over the last century (Garnaut 2011). Earth would expect warmer, wetter winters; more intense downpours of rain; hotter, drier summers, with more frequent and extreme high temperature; sea levels to rise further, with an increased risk of tidal surges. New observations of a changing climate include an increase in extreme weather events (Garnaut 2011). Recent extreme weather events have raised questions about whether the patterns and nature of these events are changing. The Black Saturday fires in Victoria in 2009 and recent major cyclones in Queensland are both consistent with expected outcomes in a warming world, although conclusions cannot be drawn about direct cause and effect (Garnaut 2011).

Extreme weather events are often short-lived, abrupt events lasting only several hours up to several days; they are 'shocks' within the climate system. Examples include extremely hot days, very heavy rainfall, hail storms, flooding and tropical cyclones. These are ‘acute’ extreme events (Climate Commission 2013). A few extreme events can last for much longer periods of time and are usually termed extreme climate events. These are ‘chronic’ extreme events (Climate Commission 2013). A heavy rainfall event is a deluge of rain that is much longer and/or more intense than the average conditions experienced at a particular location. The amount of rainfall in a day is also referred to as rainfall intensity. An extreme rainfall event may also be defined by its ‘return period’ (Climate Commission 2013). A 1-in-20 year event at a site is the daily rainfall total that would be on average expected to occur once in 20 years. The magnitude of such an event would vary from site to site (Climate Commission 2013).

1.2 IMPACT OF FLOODS IN AUSTRALIA

Flooding has been recognised globally as an issue of utmost concern and the threat of flooding poses problem to all nations in the World. Flooding in Australia has been centre stage in national and international media and forums after the January 2011 flood (Chief Scientist 2011). Floods are one of the most expensive types of natural disaster in Australia with direct costs estimated over the period 1967-2005 averaging at AUS$377 million per year (calculated in 2008 Australian dollars) (Chief Scientist 2011). Until recently, the costliest year for floods in Australia was 1974, when floods affecting New South Wales, Victoria and Queensland resulted in a total cost of AU$2.9 billion (Chief Scientist 2011). The Queensland Government estimated costs for the 2011 floods would exceed this figure for Queensland alone; with the damage to local government infrastructure estimated at AUS$2 billion, and the total damage to public infrastructure across the state over AU$6 billion (Chief Scientist 2011).

The catastrophic impacts of the flooding events that devastated central and southeast Queensland, and the destruction wreaked by Tropical Cyclone Yasi saw more than 99 per cent of Queensland declared as disaster affected (Queensland Reconstruction Authority 2013). Approximately 59 rivers flooded with 12 rivers breaking flood records and 19,000 km of state and local roads were affected by the 2010-2011 floods. Tropical Cyclone Yasi was a Category 5 cyclone and the first of that magnitude to strike the Queensland coast (Queensland Reconstruction Authority 2013). It is estimated the reconstruction and restoration of flood affected areas will cost in the order of AU$5 billion, with damage sustained from Tropical Cyclone Yasi estimated to exceed AU$800 million (Queensland Reconstruction Authority 2013).

In 2010, La Niña was exceptionally strong in Australia, as measured by the Southern Oscillation Index (Chief Scientist 2011). August to December of 2010 was very wet with the heavy rains of earlier months extending south to include most of Victoria and Tasmania (Chief Scientist 2011). Virtually all of Australia had above-average rainfall for this period and many areas had record-breaking rainfall. The rainfall for this period was the highest on record across vast areas of eastern Australia, particularly Queensland, as well as the top end of the Northern Territory, and the Gascoyne region of Western Australia (Chief Scientist 2011). Numerous monthly rainfall records were set, particularly in the central and eastern region. Flooding was a regular occurrence and there was substantial recharge in many depleted water storage systems (BoM 2010a). The most damaging floods occurred across Queensland in the final week of 2010 (continuing into early 2011). The most severe impacts were in central and southern inland Queensland. Numerous rivers throughout the region reached record levels (BoM 2010a). From December 2010 to January 2011, Queensland experienced widespread flooding, with three quarters of the state declared a disaster zone (Chief Scientist 2011).

Significant flooding also occurred across inland New South Wales in December 2010, particularly in the Murrumbidgee and Lachlan catchments (BoM 2010a). Major floods also occurred in northern Victoria and the New South Wales Riverina in early September, 2010 and in the Gascoyne River around and upstream of Carnarvon, Western Australia, in mid-December 2010 (BoM 2010a).

Heavy rainfall occurred over many parts of Queensland from 24 January to 8 February, 2010 associated with Tropical Cyclone Olga as the system weaved a path across the state. Olga made two coastal crossings and tracked from the north to the south of the state (BoM 2010b). The most intense rainfall during this period occurred over southeast Queensland where more than 400 mm of rain fell on 7 February at Clagiraba Road Alert on the Coomera River (BoM 2010b). As Olga tracked a path across Queensland over a period of at least two weeks, heavy rainfall and river flooding occurred in many parts of the state (BoM 2010b). Many areas were flooded in Yeppoon and Rockhampton on February 1, 2010.

During late January 2013, heavy rainfall in Queensland and Northern NSW associated with Ex-Tropical Cyclone Oswald caused areas of flooding (The Senate 2013). Tropical Cyclone Oswald tracked along the east coast of Australia from Mossman to Sydney from 22 to 29 January, 2013 (BoM 2013). Over, just four days, Gladstone (Queensland) received approximately 820 mm of rain, which exceeded the amount of rain they had received in the calendar years of 2011 and 2012 (BoM 2013). Major flooding devastated many areas in Queensland, extending from 22 January until 17 February, costing an estimated AU$2.4 billion (BoM 2013). The flood events and associated heavy rainfall, a result of Tropical Cyclone Oswald, had a catastrophic effect on Queensland; for example, approximately 5,845 km of State roads and 2,800 km of State rail network were closed (Queensland Government 2013). With the increasing frequency of extreme rainfall events in recent years, extreme weather will continue to pose challenges to State and Local Government Agencies.

Floods have impact on both individuals and communities, and have social, economic, and environmental consequences. The consequences of floods, both negative and positive, vary greatly depending on the location and extent of flooding, and the vulnerability and value of the natural and constructed environments they affect (Chief Scientist 2011). The floods have a significant impact on the Australian economy, in addition to world commodity and agriculture prices. Queensland accounts for approximately 20% of the Australian economy, 60% of global coking coal exports and 28% of Australia's fruit and vegetable production. The floods were expected to have a negative short-term effect on economic growth (IBISWorld 2011).

1.3 SIGNIFICANCE OF THE RESEARCH

Australia has a road network system of over 800,000 km and worth over AU$100 billion. Queensland has about 186,859 km of public roads. Stewardship of this network lies with two organisations, the Department of Transport and Main Roads (TMR), Queensland and Local Governments. The TMR manages approximately 34,000 km of State-controlled roads. These roads comprise the major traffic carrying and strategic roads in the state (TMR 2014b). The state controlled network is estimated to have a replacement value of more than AU$65.5 billion. The local government controlled network is estimated to have a replacement value of more than AU$10 billion (Talbot and Pelevin 2003). This road network is an important physical asset for the state and local governments. Chai et al. (2010) indicated that environmental and climate factors had the greatest effect on the pavement performance of the local roads in Southeast Queensland (SEQ). As a result, the impacts of extreme weather events, such as flooding, on the service life as well as design and maintenance of road pavements need to be studied.

Roads were among the assets destroyed when Queensland experienced its worst flooding in more than a century in early 2011. The design of pavements is based on moisture and temperature patterns reflecting the historical climate of the location. As a result, it is expected that over the next few years, pavements will be subjected to many different types of climatic conditions over the design life than was originally anticipated. Climate change and extreme weather events will present additional challenges to pavement engineers because of the number of implications weather conditions have on the design, construction and maintenance of road pavements. The changing pattern of rainfall, temperature, and evaporation can alter the moisture balances in the pavement foundation and accelerate the deterioration of pavements. As such, climate change and extreme weather events, such as flooding and frequent intense heavy rainfall events, will have impacts on pavement performance and will influence the rate of pavement deterioration. The effects of extreme climate on pavement deterioration can significantly influence planning and management for road maintenance and rehabilitation.

Implementation of a systematic programme is essential to investigate the deterioration of roads subject to extreme weather events, such as flooding and frequent intense heavy rainfall events. It is the main key to developing deterioration models for roads subject to flooding. Robust pavement deterioration models at a project level is crucial and will be useful for local road agencies. Hence, the direct impacts (significance) of the research are outlined as follows:

- Identification of the accelerated deterioration of pavements during the post-flood, short term rapid deterioration phase;
- Accurate prediction of the pavement condition of the post-flood short term rapid deterioration phase following a flooding event using new models; and
- Direct and practical application of the study for the strategic planning of managing the rapid deterioration of structural strength and surface conditions, such as rutting and roughness of pavements, and reducing the cost of road maintenance and rehabilitation.

This research will significantly help local road agencies plan and identify the adaptation options to build resilient road infrastructure in flood prone areas to reduce the impacts of the extreme weather events, such as flooding, on pavement deterioration. In future, the study will make a significant contribution to the preparation of cost-benefit models if building resilient road infrastructure in flood prone areas is not a feasible option for a road agency. The outcomes of this research can be used in asset management plans to identify the alternative options to manage flood affected roads.

The recent natural disasters experienced throughout Queensland and the subsequent loss of life and property and extensive damage to the State's infrastructure led to serious disruption of the State's communities, economy and environment. Many Queensland communities have been flooded every couple of years. Options or ways to build up resilience to natural disasters or ease the impacts of flooding events in those areas regularly affected must not be overlooked (Queensland Government 2013). Resilience starts with how communities view themselves. It is a mindset that recognises these natural disasters are a way of life in Queensland, that local communities need to be smart and practical in their approach to planning ahead for these events, and that they must respond together to rebuild quickly and more effectively each time an event occurs (Queensland Government 2013). Damage to key infrastructure, community facilities and the natural environment confirms the importance of an all hazards approach to disaster resilience within the Queensland disaster management framework. The Department of Local Government, Community Recovery and Resilience focuses on enhancing infrastructure and community resilience across Queensland. Building disaster resilience requires a team effort. The Department of Local Government, Community Recovery and Resilience works closely with local government, other key government agencies and asset owners to ensure that actions are focussed on community outcomes and services are joined up and efficient. Resilience starts with ensuring communities and infrastructure are built with natural hazards in mind. Community resilience, through the readiness of community members to lead and work together, is critical to the restoration and recovery of social structures (Queensland Government 2013).

Hence, this research is also significant for the community to improve the perception of road maintenance and management in flood-affected areas so that emergency funding for flood affected roads can be managed efficiently.

1.4 AIM OF THE RESEARCH

This PhD research project, commencing in early 2013, evaluated the impacts of extreme weather events, such as flooding and frequent intense heavy rainfall, on pavement deterioration. This research has two main aims which are as follows:

- Advance the knowledge and understanding of the impacts of extreme weather events, such as flooding, on pavement deterioration by assessing the damage caused by recent flooding events like the January 2011 flood and 2013 floods in Queensland and New South Wales.
- Developing mechanistic-empirical deterministic pavement deterioration models to predict the rapid deterioration of structural and surface conditions (roughness and rutting) of pavements impacted by river flooding or gradual increase of flood water.

The study examined the structural and surface condition (roughness and rutting) of flooded pavements using Falling Weight Deflectometer (FWD) and surface condition data. Data were collected from the Brisbane City Council (BCC), the Department of Transport and Main Roads (TMR), Queensland, and the Roads and Maritime Services, New South Wales (RMS, NSW).

1.5 RESEARCH GAP

Recent extreme weather events, such as heavy rainfall, major cyclones and flooding, have drawn attention to the importance of research on the impact of these events on roads in Australia. The impact of gradual rising of flood water on the pavement and subgrade is not yet clearly understood. There is still a gap in the research and literature on the assessment of damage and deterioration of inundated pavements. The procedure is complex and time consuming and involves historical data, Falling Weight Deflectometer (FWD) deflection and surface condition data collection before and after flooding on the same road section. Therefore, deflection and surface condition data are needed on partially and fully flooded roads on a regular basis for assessing deterioration. Until recently, few road authorities across Australia had collected data on such occurrences.

A number of unknowns exist in regards to the aftermath of flooding events on pavements, for example:

- The impact of gradual rise of flood water on the pavement and subgrade as they become saturated at that time
- The impact of allowing traffic to travel on roads immediately after flood waters recede
- The impact of heavy trucks, removing debris following flood events, on a lightly trafficked road, which was not designed to carry such loads under these conditions.

Hence, it is imperative to investigate whether flooding events accelerate deterioration of the pavements. This result is directly linked to the future cost of repair of these roads.

1.6 RESEARCH QUESTIONS

A set of research questions were identified which are:

1. What are the necessities of specific pavement deterioration models for roads subjected to flooding?
2. What is the rate and trend of pavement deterioration before and after inundation?
3. What are the long-term consequences of flooding on the inundated pavements?

Research question one is partly answered in Chapter one (Introduction), and in Chapters two and three (literature review). Currently, there is no suitable road deterioration model to evaluate the rapid deterioration of pavements after flooding.

Research questions two and three are addressed in Chapters five and six with the assessment of flood affected pavements of Brisbane City Council, TMR, Queensland and RMS NSW. Analysis of the rate and trend of deterioration of pavements and long term consequences of flooding on inundated pavements are also included in Chapters five and six.

1.7 OUTLINE OF THE THESIS

This thesis consists of seven chapters and a reference section along with appendices. The organisation of the dissertation is briefly described below.

Defining the Problem

Chapter 1 provides an introduction to the impact of extreme weather events. This chapter also presents the significance, research gap, aims and methodology of the research.

Literature Review

The literature review has been presented in two chapters. **Chapter 2** discusses the literature related to pavement deterioration. It reviews the literature on falling weight deflectometer (FWD), backcalculation of FWD data, considerations for developing a deterioration model, general approach to predict pavement performance, software used for backcalculation, and the impact of water and moisture content on pavement. It also presents a review of recent deterioration models in Australia and other parts of the world which are used to quantify the deterioration of structural and surface conditions of pavements.

Chapter 3 reviews the previous work carried out by other researchers in Australia and globally on the effect of floods on pavements.

Methodology and Data Collection

Chapter 4 discusses research methodology for data collection and analysis. It also justifies the data collection process, data collected, data preparation and sampling of data.

Analysis of Data

Chapter 5 presents the assessment of the structural conditions of the flood affected pavements of Brisbane City Council and RMS, NSW. It contains statistical analysis and development of the structural deterioration models for flood affected pavements.

Chapter 6 presents the assessment of the surface conditions, such as rutting and roughness, of the flood affected pavements of TMR, Queensland. It contains statistical analysis and development of the deterioration models for rutting and roughness.

Conclusion and Recommendation

Chapter 7 summarises the contributions, findings and recommendations for further research.

2. LITERATURE REVIEW

2.1 PAVEMENT MANAGEMENT SYSTEM

Pavement management in its broadest sense encompasses all the activities involved in planning, design, construction, maintenance, evaluation and rehabilitation of the pavement. A pavement management system (PMS) is a set of tools or methods that assists decision makers in finding optimum strategies for providing, evaluating and maintaining pavements in a serviceable condition over a given period of time. The function of a PMS is to improve the efficiency of decision making, expand its scope, provide feedback on the consequences of decisions, facilitate the coordination of activities within the agency and ensure consistency of decisions made at different management levels within the same organisation. It is convenient to describe pavement management in terms of two generalised levels: (1) the network level management sometimes called the program level, where key administrative decisions that affect the programme for road networks are made; and (2) the project management level, where technical management decisions are made for specific projects. Considering the needs of the network as a whole, a total PMS provides a comparison of the benefits and costs for several alternative programs, making it possible to identify the best option (AASHTO 1993).

A PMS collects and monitors current pavement information, forecasts future conditions, and prioritises alternative reconstruction, rehabilitation, and maintenance strategies to achieve a steady state of system preservation at a predetermined level of performance. Such systems are strongly dependent upon the confidence level of the prediction of future pavement condition. Hence, the pavement performance model is a key component of the PMS to predict the future performance of pavements. It is also a requisite for optimising maintenance, rehabilitation, and reconstruction (MR&R) policies over a planning horizon (Meegoda and Gao 2014). The accurate prediction of pavement performance is important for efficient management of the transportation infrastructure. By reducing the prediction error of pavement deterioration, agencies can obtain significant budget savings through timely intervention and accurate planning (Madanat 1993). Selecting the appropriate rehabilitation strategy for any flexible pavement is simple if the mechanism causing pavement deterioration is known. Unfortunately, identifying and verifying subsurface pavement deterioration mechanisms is extremely difficult (Scullion and Saarenketo 1999).

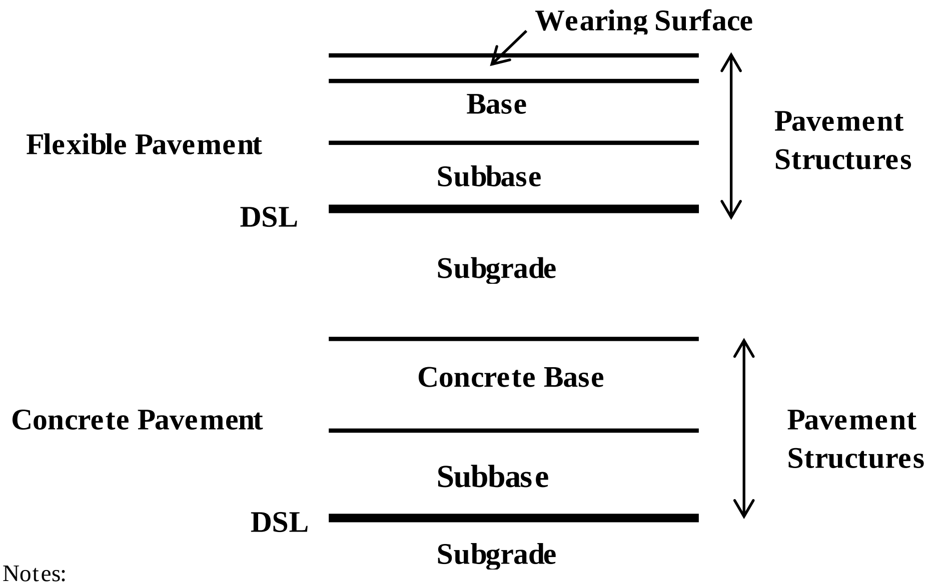
Future performance of pavements depends on existing pavement condition and other variables controlling pavement deterioration, such as truck traffic volume, climate, and pavement structure. By reducing the prediction error of pavement deterioration, transportation agencies can obtain significant budget savings through timely intervention (Prozzi and Madanat 2004). At the project level, adequate activity planning and project prioritisation requires estimation of future pavement performance. Predicting pavement performance is important to establish the specific corrective actions needed, such as maintenance and rehabilitation at the project level (Prozzi and Madanat 2004). At the network level, pavement performance prediction is essential to develop a rational budget for resource allocation (Meegoda and Gao 2014; Prozzi and Madanat 2004). At the network level, planning, budget justification, and resource allocation are dependent on the predicted deterioration of the overall facilities. At the project level, determining when an existing section will become deficient is directly related to the planning process for scheduling maintenance and rehabilitation work (Hong and Prozzi 2010).

This chapter reviews the literature on the evaluation, assessment and modelling of the deterioration of structural and surface condition of pavements to understand the mechanism of modelling deterioration of pavements. This chapter also discusses different types of pavements, causes of pavement deterioration, evaluation of pavement performance and measuring the structural deterioration of pavements using falling weight deflectometer (FWD) testing and general approaches to predict the performance of pavement. Also included in this chapter is a review of the literature on existing road deterioration models. This is necessary to determine what is known and what is unknown to develop a deterioration model for pavements subjected to extreme weather events, such as flooding.

2.2 TYPES OF PAVEMENT

Pavements are divided into two main categories namely flexible and rigid. The flexible pavements may consist of a relatively thin wearing surface built over a base course and subbase course, and they rest upon the compacted subgrade. In contrast, rigid pavements are made up with portland cement concrete slab and may or may not have a base course between the pavement and the subgrade (Yoder and Witczak 1975). This study investigated flood affected flexible pavements in Queensland which are the most common types of pavement in Australia.

In Australia, pavements are assumed to be constructed to the usual quality standards specified by Austroads member authorities (Austroads 2012). Components of flexible and rigid road pavement structures are shown in Figure 2.1 (Austroads 2012).



Notes:

1. DSL= Design Subgrade Level
2. Base and Subbase may contain more than one layer
3. Wearing course of a flexible pavement may be asphalt or bituminous seal
4. In a rigid pavement the concrete base may be surfaced with an asphalt wearing course
5. An imported subgrade or selected subgrade material may be placed over the natural subgrade.

Figure 2.1: Components of flexible and rigid road pavement structures (Austroads 2012)

Pavement type varies markedly with the function of the road, traffic loading, availability of materials, and the environment. Lightly-trafficked roads usually comprise unbound granular pavements with thin bituminous surfacings. Where an asphalt surfacing is provided, it is common for the thickness of asphalt to be 25 - 45 mm. More heavily trafficked roads may require the asphalt to extend to more than the surface layer, with the asphalt commonly supported by a granular subbase. Some heavily-trafficked roads (e.g. freeways) have high level performance requirements and need to be designed to minimise traffic delays due to road maintenance during their service lives. Such pavements commonly have a design traffic loading exceeding 1×10^7 ESA (Equivalent Standard Axles) and are sometimes referred to as ‘heavy-duty’ pavements (Austroads 2012). The following pavement types, often used in conjunction with high material standards, may be considered heavy-duty pavements:

- deep strength asphalt, with thick asphalt on cement stabilised granular subbase
- flexible composite, comprising a thick asphalt on lean concrete subbase
- full depth asphalt
- unbound granular base with sprayed seal surfacing
- jointed plain (unreinforced) concrete pavements
- jointed reinforced concrete pavements
- continuously reinforced concrete pavements.

Such heavy-duty pavements are commonly supported by higher strength, stable materials consisting of granular subbases and/or selected subgrade materials (Austroads 2012).

Granular Pavements with Sprayed Seal Surfacings

Unbound granular pavements with sprayed seal surfacings are the major pavement type in rural Australia, comprising some 90% of the length of all surfaced roads. They comprise the majority of light and moderately trafficked rural roads and have also been successfully used on heavily-trafficked roads, subject to suitable materials, environments and construction and maintenance standards. This pavement type is extensively used due to its low initial cost (Austroads 2012).

Cemented Granular Bases with Sprayed Seal Surfacings

The use of cemented bases with sprayed seal surfacings is more commonly associated with the rehabilitation treatments of granular pavements than new construction works. With the exception of temporary pavements, this pavement type is seldom used for new works due to significant performance issues associated with shrinkage cracking (Austroads 2012).

Granular Pavements with Thin Asphalt Surfacings

Unbound granular pavements with single thin asphalt surfacings are structurally similar to sprayed seal pavements except that asphalt surfacing may fatigue crack. For this pavement type the asphalt surface makes little contribution to the overall strength of the pavement but provides greater resistance to minor traffic damage as well as a smoother and more durable surface. These attributes make it particularly suited to residential streets and other light traffic urban applications where risk of fatigue cracking is lower. With suitable quality of materials and construction standards, these pavements are sometimes used for urban collector and occasionally main roads, although they may not provide the same serviceability as more heavily bound pavements. Some jurisdictions restrict the use of single layer asphalt surfacings on new pavements because of the performance risks associated with them (Austroads 2012).

Thin asphalt surfacing can also be used on light to moderately trafficked rural road pavements, where sprayed seals do not provide adequate serviceability, and where the risk of fatigue cracking is acceptable, e.g. intersections and other areas of turning traffic, or to provide improved ride quality (Austroads 2012).

Asphalt over Granular Pavements

These pavements comprise multiple asphalt layers over a granular base and/or subbase. In these pavements the purpose of the asphalt layers is to provide a wearing surface and to make a significant contribution to the structural capacity of the pavement. Where the asphalt thickness is less than 150 mm, the granular base layer(s) provides a substantial proportion of the load carrying capacity and both deformation and fatigue distress mechanisms are possible. Therefore, the asphalt and granular base materials must be of appropriate quality to ensure the intended service life results (Austroads 2012).

The main application for asphalt on granular pavement is on medium traffic urban roads. It may also be suitable for rural highways and main roads depending on actual climate and traffic loads. Moisture retained in asphalt surfacing can increase the risk of moisture damage to the underlying asphalt. Thick asphalt pavements usually incorporate 10 mm or 14 mm dense graded asphalt over the base asphalt/intermediate layers. This dense graded asphalt can inhibit moisture entry if its properties and construction are carefully managed. Trafficking of the dense graded asphalt can also assist in decreasing surface permeability, although it also increases the risk of moisture ingress in the short term due to increased exposure. A heavy tack coat or sprayed seal (depending on local practice) may be required before the surface asphalt is placed to ensure adequate waterproofing (Austroads 2012).

Flexible Composite, Deep Strength and Full Depth Asphalt Pavements

In this case, asphalt is used in both the surface and bound base layers to provide a significant proportion of the load carrying capacity. Deep strength asphalt pavements may also incorporate a cemented or lean concrete subbase. Granular subbases and/or selected subgrade materials may be provided under the bound layers to provide a working platform. These pavements are suited to moderately to heavily trafficked roads, primarily in urban areas (Austroads 2012).

2.3 FALLING WEIGHT DEFLECTOMETER

The non-destructive testing (NDT) equipment used in making the measurements includes a variety of modes for applying loads to a pavement and a number of sensors for measuring the pavement response (Lytton 1989). The loading methods include: Static or slowly moving loads, Vibrations, "Near field" impulse methods, and Wave propagation methods. Output responses are measured on the surface or with depth below the surface. Surface measurements are made with geophones that sense the velocity of motion, accelerometers and linear variable differential transformers (LVDT) that measure displacement (Lytton 1989).

The falling weight deflectometer (FWD) test is the most widely accepted and used technique for non-destructive evaluation of pavements. It is routinely used by pavement engineers to evaluate in situ pavement layer moduli. The objective of the test is to excite the pavement by impact of a falling weight and measuring its response under the load levels equivalent to those applied on a pavement by traffic. The pavement response to the impulse load is then analysed or back-calculated to evaluate the in situ pavement layer moduli (Hadidi and Gucunski 2010; Meier 1995). With the increasing need for economical non-destructive testing (NDT) methods, the FWD test has emerged as a viable way to estimate the pavement structural adequacy while leaving the pavement section intact and traffic largely unaffected (Westover and Guzina 2007). During the FWD test, the pavement deflection response is measured by transducers at different offsets from the load. The maximum pavement displacements at transducer locations, collectively referred to as the deflection bowl (or deflection basin) or the displacement time histories at each receiver location, are then reported as pavement response. With pavement layer thicknesses as a given input, the measured pavement response is then analysed or back-calculated to infer the in situ pavement layer elastic moduli. The back-calculated pavement moduli are then used to design overlays, estimate remaining life of a pavement, evaluate the load transfer efficiency of joints in rigid pavements, identify weak areas in the pavement structure, and perform network level monitoring (Hadidi and Gucunski 2010).

Figure 2.2 represents the generalised deflection bowl of a falling weight deflectometer testing (BCC 2011), while Figure 2.3 shows the use of FWD for pavement strength testing in the outer wheel path, in order to estimate structural capacity for the purposes of overlay design (Fugro 2012).

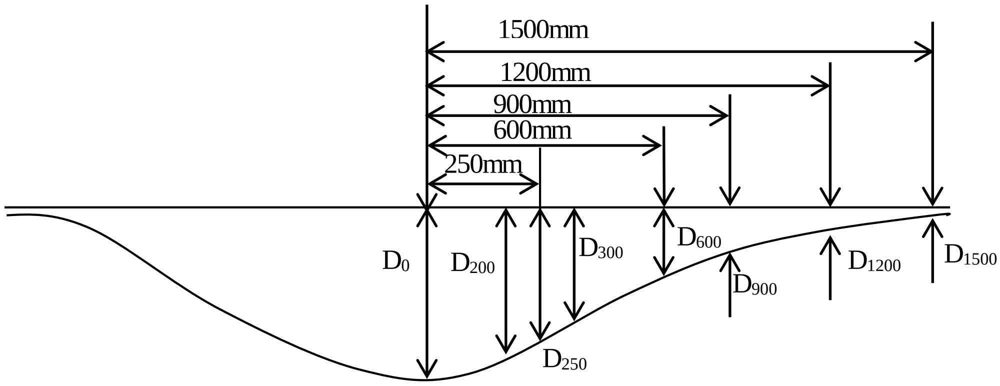


Figure 2.2: Generalised FWD deflection bowl

Figure 2.3: Using FWD for pavement strength testing in the outer wheel path (Fugro 2012)

A typical FWD has a load capacity in the order of 120 or 150 kN (Austroads 2005). Table 2.1 shows target FWD test loads and corresponding surface stresses (Austroads 2005).

Table 2.1: Target FWD test loads and corresponding surface stresses (Austroads 2005)

Target Test Load (KN)	**Corresponding surface stress (kPa)**	**Rounded surface stress (kPa)**
35	495	500
40	566	550
50	707	700

It is also necessary to ensure that target test loads are selected appropriately to assess the strength and stiffness of the pavement. Stresses of 550 kPa and 700 kPa are typical FWD targets, and actual test load stresses need to be within 15% of the relevant target to achieve a reasonable accuracy within the deflection analysis (Austroads 2005).

2.3.1 Use of Deflection Testing Data

Deflection data can provide significant information about the state of a pavement (Austroads 2011). For instance:

- Very high local deflections (more than 1.5 mm) may indicate weak subgrade conditions.
- High values of curvature function may indicate low stiffness in the upper pavement layers, or a pavement with cracked surfacing. Substantially different deflections and curvature functions between left and right wheel paths may indicate the presence of a pavement widening, patches, water ingress from the adjacent verge or seasonal moisture effects.
- A high deflection peak near a pavement edge may be caused by poor local drainage, such as a blocked subsurface drain.
- A series of high deflection peaks at a similar location in both test wheel paths may indicate a poorly backfilled culvert or service trench or a poorly drained junction between two pavement types.
- A generally low but extremely variable deflection pattern may indicate an old, failing, bound pavement which may be cracked or poorly patched.
- A substantial (> 0.15 mm) positive residual Benkelman Beam deflection implies a weak pavement, probably due to poor compaction. A negative residual deflection may indicate shearing within the pavement, but is more commonly associated with pavements incorporating cemented layers where the beam supports are within the deflection bowl. A well-defined area of low deflections with high positive residual Benkelman Beam deflections may indicate unstable pavement material.

2.3.1.1 Deflection Ratio

For sealed granular pavements, bowl deflections are measured and typically reported at distances of 0, 200, 300, 450, 600, 900 and 1500 mm from the centre of the impulse test load. These deflections are referred to as D_0, D_{200}, D_{300}, D_{400}, D_{600}, D_{900} and D_{1500}, respectively. The deflection D_0 represents the point of maximum deflection under the impulse load, and is used in estimating network pavement strength. The D_{200} and D_{300} values are used to estimate D_{250} which is used in calculating the deflection ratio, DR (refer to Equation (2.1) to eliminate cemented base pavements from the estimate of network pavement strength because of the different performance characteristics of these pavements.

$$DR = \frac{D_{250}}{D_0} \tag{2.1}$$

This ratio is used to indicate whether sections of the pavement are bound, unbound or excessively weak to the given traffic load. Indicative values for the deflection ratio are shown in Table 2.2 (BCC 2011). A deflection ratio value of:

- greater than 0.8 would indicate a rigid or bound pavement
- between 0.6 and 0.7 would be expected for a good quality unbound pavement
- less than 0.6 would indicate a possible weakness in the pavement (BCC 2011).

Table 2.2: Indicative values for deflection ratio (BCC 2011)

Pavement Type	**Deflection Ratio**	
	Range	**Mean**
Granular* (normal or second standard)	0.50 – 0.70 0.50 – 0.80	0.60 0.65
Asphalt (full depth)	0.70 – 0.90	0.80
Cemented material* (full depth)	0.85 – 0.95	0.90
Granular* on cemented	0.70 – 0.90	0.80
Asphalt on granular	0.60 – 0.85	0.70
Asphalt on demented	0.85 – 0.95	0.90
Asphalt on granular on cemented	0.70 – 0.90	0.85
*with thin bituminous surfacing		

The D_{600}, D_{900} and D_{1500} values are used in estimating the Adjusted Structural Number (SNP). The additional cost of recording and storing D_{450} can mostly be warranted on the basis of the potential to use this data in more detailed analysis (Austroads 2005).

Deflections are desirable as far as possible from the centre of the applied load and preferably beyond the 900 mm offset mentioned above, because deflections at large offsets increase the likelihood of recording the full extent of the bowl. FWDs are capable of measuring deflections at offsets up to 2,400 mm.

2.3.1.2 Curvature function

The shape (curvature) of the deflection bowl is used to estimate the likelihood of fatigue cracking in an asphalt layer. The curvature is defined by the Curvature Function (CF), given as follows by Equation (2.2) (TMR 2012).

$$CF = D_0 - D_{200} \tag{2.2}$$

where,

CF	=	curvature function
D_0	=	maximum rebound deflection
D_{200}	=	deflection at a point 200 mm from the maximum rebound deflection

For granular pavements with thin bituminous surfacing, the curvature function is likely to be 25% to 35% of the maximum deflection. Values higher than about 35% may indicate that the granular base course has low strength. High values of the CF (e.g. 0.4 mm for results derived using a FWD with a 40 KN loading, the Benkelman Beam or PAVDEF) may indicate a pavement that is lacking stiffness, a very thin pavement, or a pavement with a cracked asphalt surface. Low values of the CF (e.g. <0.2 mm for results derived using a FWD with a 40 KN loading, the Benkelman Beam or PAVDEF) indicate a stiff pavement (Austroads 2011; TMR 2012).

2.3.1.3 Subgrade response (D_{900})

The deflection at a point 900 mm from the point of maximum deflection is referred to as the D_{900} value. For pavements without bound, thick asphalt or rigid layers, the D_{900} value has been found to reflect a subgrade response that remains essentially unaffected by the structure of the overlying pavement and has been used to estimate the subgrade California Bearing Ratio (CBR) at the time of testing (TMR 2012).

2.3.1.4 Using D_{450} for Thin Bituminous Pavement to Estimate CBR

D_{450} can be used to measure the subgrade response of thin bituminous pavements with thickness of the asphalt layer varying from 30 to 50 mm and granular layer varying from 160 to 250 mm (Chai et al. 2013). Equation (2.3) was developed by Chai et al. (2013). Brisbane City Council uses this method to estimate the subgrade CBR (Chai et al. 2013)

$$CBR_{Subgrade} = 2.6523\,(D_{450})^{-1.001} \tag{2.3}$$

where,

$CBR_{Subgrade}$ = Subgrade CBR value (%)

D_{450} = deflection at a point 450 mm from the maximum rebound deflection

2.3.2 Temperature Correction

Deflection and back analysis results must be corrected to the average working temperature of the pavement for a particular location. This average working temperature is referred to as the Weighted Mean Annual Pavement Temperature (WMAPT) (TMR 2012). For design expediency, WMAPT zones have been derived for the state as given in Chapter 2, page 2-47 of the Pavement Rehabilitation Manual (TMR 2012). Temperature correction for Benkelman Beam and normalised 40 kN FWD deflections results for a pavement with a thin asphalt surfacing or seal is shown in Figure 2.4. For Brisbane, WMAPT ranges from 29° to 32° Celsius (TMR 2012).

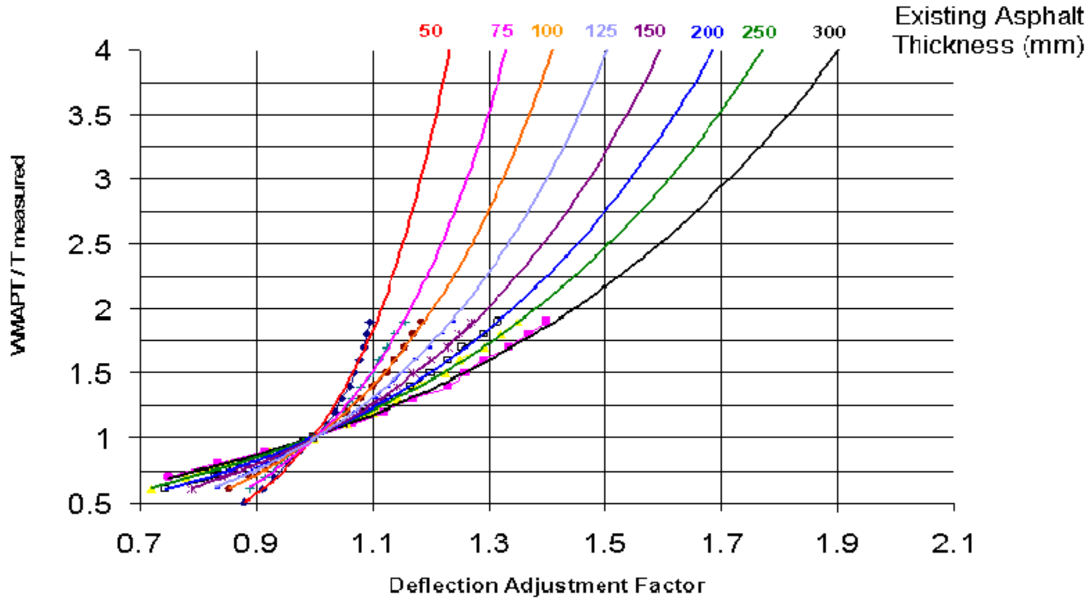


Figure 2.4: Temperature correction for Benkelman Beam and normalised 40kN FWD deflections results for a pavement with a thin asphalt surfacing or seal (TMR 2012)

The temperature of the asphalt at a depth of 25 mm should be measured at regular intervals and when weather conditions change. The temperature at this depth is considered to be equivalent to the average temperature throughout the layer. The frequency at which the temperature is measured needs to be carefully considered. More frequent measurements are required when:

- the temperature is changing quickly; or

- thick asphalt layers exist as such layers are more highly influenced by the asphalt temperature.

Where asphalt temperatures exceed 60°C, testing should cease as the results obtained become unreliable. When designing using the deflection reduction method, individual D_0 and CF results are adjusted to correct for the difference in performance between the measured field temperatures and the WMAPT (TMR 2012). The design steps for asphalt surfaced granular pavements with a total thickness of asphalt of 50 mm or more are:

- Determine the ratio of WMAPT for the site to the measured temperature at the time of testing.
- Determine the correction factors from Figure 2.4 as appropriate.
- Multiply the deflections (D_0) and curvatures (CF) by the corresponding correction factors.

For pavements with asphalt with a total thickness less than 50 mm, no temperature correction is required. To adjust measured deflections for temperature, the existing asphalt thickness is required. Asphalt thicknesses may be obtained from historical data, measuring pavement cores, measurement taken during trenching (or excavation of test pits) or GPR results (that are accurately calibrated against cores, test pits or trenches) (TMR 2012).

2.3.3 Moisture correction for FWD Testing

If possible, FWD deflection testing should be undertaken when the subgrade is in the weakest condition, normally the wet season. Ideally, as correction factors are influenced by things such as subgrade type, rainfall, location of water table and pavement types, they should be developed from studies conducted by each Region or District. Where this is not possible, guidance is provided in Table 2.3. The values in Table 2.3 are for use when doing designs based on the deflection reduction method. To correct for moisture, the D_r and representative D_{900} obtained from testing in a particular season are multiplied by the correction factor given in Table 2.3. The corrections are not applied to individual results (TMR 2012).

Table 2.3: Seasonal moisture correction factors for a pavement with a thin asphalt surfacing or seal (TMR 2012)

[1]**Pavement condition**	[2]**Correction factor for TMR Regions/Districts**			
	For deflections measured during the end of wet season	For deflections measured during the end of dry season		
	All Regions / Districts	Rockhampton District, Metropolitan, North Coast, South Coast and Wide Bay / Burnett Regions (includes Brisbane, Bundaberg, Gold Coast, Gympie, Ipswich and Logan, Moreton and Sunshine Coast Districts)	Emerald District, Darling Downs, Central West North West and South West Regions (includes Toowoomba and Warwick Districts)	Far North, Mackay / Whitsunday and Northern Regions
Weak pavements (i.e. D_r > 1.5 mm)	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.4)[3]
Intermediate pavements (i.e. 1.5 mm > D_r >0.9 mm)	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.5)[3]
Strong Pavements (i.e. D_r <0.9 mm	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.6)[3]

Notes:
[1] D_r values quoted in this table are for deflection results from Benkelman Beam, PAVDEF or FWD testing with a 40 KN load.
[2] Applied only to wheel paths other than outermost wheel paths. In situations where the water table is within 1 m of the subgrade level throughout most of the year, no correction should be applied.
[3] Values in brackets apply for silty and clayey silt subgrades where greater variation in deflection level may be expected.

Generally, the Outer Wheel Paths (OWPs) are susceptible to environmental forces while the moisture of the other wheel paths is generally relatively constant. For this reason, moisture correction factors are only applied to the deflections measured in wheel paths other than the outer wheel path, to attempt to stimulate the anticipated weakest condition of the pavement (TMR 2012). The factors given in Table 2.3 are for guidance in situations where more reliable information is not available (TMR 2012). When comparing outer wheel paths (OWP) deflections to moisture corrected deflections in other wheel paths the following should be noted:

- If the corrected deflection/s are higher than corresponding OWP deflection/s, then the corrected deflection/s should be used to determine D_r.

- If the corrected deflections are lower than the corresponding OWP deflection/s, a check should be made to ensure other factors are not controlling the OWP deflection (e.g. 'box type' construction trapping moisture or providing inadequate lateral restraint). If there are no obvious defects in the OWP that would cause high deflections then the OWP deflections should be used to determine D_r.

2.3.4 Backcalculation Process of FWD Data

FWD back-calculation is mathematically an inverse problem that can be approached deterministically or probabilistically (Hadidi and Gucunski 2010). NDT and back-calculating pavement material moduli are well accepted procedures for the evaluation of the structural capacity of pavements. Back-calculation is a complex iterative procedure to determine the modulus of each pavement layer. In order to back-calculate reliable moduli and estimate their variability, it is mandatory to conduct several deflection tests at different locations along the road sections having the same layer thicknesses. At network level at least five tests and at the project level more than ten tests, for each design unit are required (Uzan et al. 1989).

2.3.4.1 Back-calculation Categories

The back-calculation procedures can be separated into several categories, depending on the type of load representation (static versus dynamic) and on the type of material characterisation (linear versus nonlinear) for viscoelastic and or plastic materials. A discussion based on the following five categories is presented in this section (Uzan 1994);

i) Static linear elastic;

ii) Static nonlinear elastic;

iii) Dynamic linear using frequency domain fitting;

iv) Dynamic linear using time domain fitting;

v) Dynamic nonlinear analysis.

Static Linear Elastic Back-calculation

In the simplest case which is widely used today, the load is assumed to be static and the material is assumed to be linear elastic. In this case, only the peak load and peak surface response deflections are used in the back-calculation (Uzan 1994).

Static Non-linear Back-calculation

In the static non-linear elastic back-calculation, only the peaks of the loads and surface deflections are used. This is in contrast with the linear scheme where all load levels are used simultaneously. The non-linear elastic back-calculation still assumes that the permanent deformation is small compared to the resilient one. In other words, the state of stress within the pavement structure is low relative to the ultimate strength. This situation applies to pavements with a moderate to thick AC layer, not with thin surfacing of less than 50 mm. In these cases, elastic theory should be replaced by the non-linear elasto-plastic theory. This will account for any permanent deformation and stress redistribution caused during loading as compared to the elastic behaviour (Uzan 1994).

Dynamic Linear Back-calculation

The dynamic back-calculation applies to the NDT equipment that applies either a steady state vibratory load or an impact load. When several responses at different frequencies are measured, each set of data may be analysed separately to obtain separate sets of moduli for each frequency and load level. Alternatively, it is possible to normalise the response with respect to load level and analyse all frequencies simultaneously to get one set of moduli (independent of frequency) (Uzan 1994).

In the case of an impact load, two approaches may be used:

i) Using frequency domain fitting;
ii) Using time domain fitting.

Dynamic Non-linear Back-calculation

The dynamic non-linear back-calculation formulation follows the previous static non-linear and dynamic linear back-calculations. As the material is non-linear, the frequency domain transformation is not applicable (Uzan 1994).

2.3.5 Regression Analysis for FWD Back-calculation

Back-calculation of FWD data often results in unrealistic layer moduli. The modulus of subgrade may be two to three times the expected value, and the modulus of an intermediate granular material may be lower than the subgrade modulus. If stresses or strains measured in the pavement are compared with theoretical values, the agreement is often poor. All theoretical models for calculating pavement response are based on a number of simplifications with respect to reality and must be verified experimentally.

Most models assume that all pavement layers consist of linear elastic materials (Ulidtz 1999). However, Ulidtz (1999) stated that treating subgrade as a nonlinear elastic material, can result in a much more realistic moduli and a much better agreement between measured and calculated stresses and strains.

Almost all the theoretical methods presently used for calculating stresses, strains and displacements in pavement structures are derived from solid mechanics and are based on the following three assumptions:

i) Equilibrium (mostly static);
ii) Compatibility (or continuity);
iii) Hook's law (linearity between stress and strain).

Based on the assumptions, a fourth-order differential equation can be established and solved, using different numerical methods, when boundary conditions are known. Unfortunately, none of the assumptions are correct for pavement structures. The loads are mostly dynamic, not static, the materials are not solid (or continuous), some are even granular, and the deformations are not purely elastic, but also plastic, viscous and viscoelastic, and the strains are mostly nonlinear functions of the stress condition. The assumptions made with respect to the boundary conditions, such as layers of infinite horizontal extent or continuity over interfaces are, in most cases, also incorrect (Ulidtz 1999).

With the Finite Element Method (FEM) it is possible to make assumptions that are closer to the reality of pavement materials and structures, but the Finite Element Method is still based on the assumption of solid (continuous) material (Ulidtz 1999).

If the subgrade has been treated as a nonlinear elastic material, with the format (Ulidtz 1999):

$$E_m = C \times \left(\frac{\sigma_1}{p}\right)^n \tag{2.4}$$

where,

E_m	=	modulus at a point of the material where the major principal (dynamic) stress is σ_1
C, n	=	constants (n is negative)
p	=	a reference stress, normally atmospheric pressure (0.1 MPa)

Even when a method is based on simplified assumptions, however it may still produce results that are reasonably correct. The only way to find out whether a particular method produces useful results is by comparing the response predicted by the method to the response measured in actual pavement structures. From such comparisons in a full scale road testing machine and from in situ test sections, it has been found that useful results can be obtained with methods where the subgrade is treated as a nonlinear material, with a modulus, E, that varies with the major principal stress, σ_1, according to Equation (2.4) (Ulidtz 1999).

Moduli back-calculated from FWD tests, using linear elastic theory, are often unrealistically high for the subgrade and unrealistically low for an intermediate granular layer. If the vertical strain is measured at the top of the subgrade, it is often found to be two to three times larger than the theoretical value (Ulidtz 1999). Most of the assumptions on which the theoretical models are based, are incorrect for pavement materials and structures. For back-calculation purposes, one of the most important deviations from linear elastic theory is the nonlinearity (or apparent nonlinearity) of the subgrade. With linear elastic theory, it is often possible to obtain a very close fit between measured and calculated deflections, especially if the pavement consists of many layers (i.e. more than 3 to 4 layers) and if a rigid bottom is included (Ulidtz 1999).

If the subgrade is allowed to be nonlinear elastic; much more reasonable moduli are usually obtained, the match between theoretical and measured stresses and strains normally improves. The finite element method may be used as the response model for pavements with nonlinear materials, but the method of equivalent thicknesses (MET, Odemark's transformations combined with Boussinesq's equations) has been found to better match the measured response in many cases, provided that subgrade is treated as a nonlinear elastic material (Ulidtz 1999).

2.3.6 Influence of Statistical Variation in FWD Pavement Analysis

In a typical pavement improvement project, a stretch of highway is divided into a number of representative pavement sections with similar pavement and traffic loading characteristics. Among other factors, important parameters such as age, construction data, layer thickness and pavement conditions are often used to determine the extent of representative pavement sections. Within a representative pavement section, FWD tests are carried out at an interval that typically varies from 500 to 1000 ft (approximately 152 m to 305 m). The pavement layer properties near an FWD test location are assumed to be uniform. A deviation from this assumption will affect back-calculated moduli and thus introduce error in the subsequent pavement analysis procedures that use FWD-based results (Siddharthan et al. 1991).

2.3.7 Sources of Errors in the Back-calculation of Layer Moduli

Sources of errors can be systematic and random. Most of the random errors are associated with the measuring devices or pavement structure geometry and condition. Most of the systematic errors are associated with the load representation, theoretical modelling of materials and analysis (Uzan et al. 1989). These systematic errors are: pressure distribution on the loaded area; pavement structure geometry and condition; material modelling; and analysis.

2.3.8 Software used for Back-calculation of FWD Data

Various software are used for back-calculation, such as CIRCLY 5.0 (Wardle 2009), ELMOD 5.0 (ELMOD 5 2006), EVERCALC 5.0 (Everseries Pavement Analysis Programs 2005), EFROMD2 (Vuong 1992) and finite element analysis. This research uses CIRCLY 5.0 as it is a well-known software in Australia. It is a powerful package that analyses a comprehensive range of load types acting on layered elastic systems. CIRCLY is supplied with a comprehensive range of published performance models. It has special features for the convenient mechanistic analysis and design of pavements using state-of-the-art material properties and performance models. CIRCLY is an integral component of the Austroads Pavement Design Guide that is widely used in Australia and New Zealand. The system calculates the cumulative damage induced by a traffic spectrum consisting of any combination of vehicle types and load configurations. Because the contribution of each vehicle/load configuration can be explicitly considered, it is not necessary to approximate multi-wheel configurations by 'equivalent' single loads.

2.3.9 Necessity of FWD Testing for Structural Deterioration

Many pavement performance characteristics (roughness, cracking, rutting, etc.) do not give an accurate assessment of the structural condition of a pavement because they mainly assess surface conditions and not structural conditions (Eijbersen and Van 1998). These surface condition measures are inexpensive to collect and have traditionally been used to broadly identify suspect areas of the network for detailed structural testing and assessment at a detailed project level (Austroads 2003).

Pavement strength can be estimated from surface deflection data, though Paterson (1987) notes that deflection measures stiffness rather than strength. Pavement strength is defined as the ability of a pavement structure to resist the traffic vehicle wheel loads that are applied to it, while pavement stiffness is defined as the resistance to deflection of the pavement structure. Pavement strength is often seen as synonymous with the structural capacity. The difference between strength and stiffness is particularly important when assessing flexible pavements that vary in their mechanism of failure. The magnitude of the measured deflections (and hence stiffness) of pavements with cemented and unbound bases would usually be significantly different. Each also has a vastly different relationship between stiffness and the structural performance that relates to pavement strength. To ensure the validity of the granular pavement strength determinations, which are based on deflection testing data, pavements with cemented bases should be excluded from the survey or the analysis. In practice, selective network testing can be difficult when the pavement configuration details of many road segments are unknown (Austroads 2003).

The FWD testing plays an important role in evaluating the structural condition of existing pavement and in understanding the effects of seasonal variations on pavement response (and subsequently on pavement performance) (ASTM 2000). The FWD testing effectively simulates actual truck loading conditions. The data generated from FWD testing have been used in different applications: back-calculation of in situ material properties; delineation of pavements subsections; and the evaluation of structural layer coefficients. Even though a large number of road agencies use the FWD to conduct these analyses, each agency has developed its own set of procedures and methods (Sebaaly et al. 1999).

The strength determination and performance characteristics of bound and unbound pavements differ significantly. For those road networks that include pavements of both types, it is recommended that the deflection data relating to sealed unbound granular pavements excludes the lengths of pavement where the maximum deflection, D_0, is less than 0.35 mm and the D250/D0 ratio is greater than 0.8.

2.4 STRUCTURAL PERFORMANCE EVALUATION OF PAVEMENT

The objective of structural evaluation of pavement is to determine the present structural condition of a pavement and its capacity to withstand traffic and environmental forces over the design period. It is also the foundation for the design of any appropriate (structural) rehabilitation treatments that may be required (TMR 2012). A sound pavement evaluation will:

- enable the designer to assess the existing pavement and determine its current condition;
- identify the causes/mechanisms of any observed pavement distress, if any;
- ascertain whether the existing pavement must be rehabilitated to withstand the predicted conditions for the required design period; and
- provide the foundation for identifying what treatments/interventions are required, if any.

The structural performance of a pavement relates to its physical condition, such as occurrence of cracking, faulting, ravelling, or other conditions which would adversely affect the load-carrying capability of the pavement structure or would require maintenance (AASHTO 1993). As structural capacity cannot be determined directly, a number of indirect indicators are used. These include (TMR 2012):

- Defects observable during a visual survey (e.g. rutting, cracking and shoving);
- Structural response to load (i.e. deflection data);
- Properties of pavement and subgrade materials (e.g. as determined from field sampling and laboratory testing); and
- Moisture related defects (TMR 2012).

The assessment of current structural capacity, and therefore the remaining structural capacity, has to be made relative to a definition of when the terminal structural condition has been, or will be, reached. The actual terminal structural condition and its associated distresses will depend on the levels of service, or functionality, required of the pavement. These levels of service and their limiting distress values are based on avoiding rapid or catastrophic failure and its consequences. This means that stricter distress limits are maintained for heavily trafficked pavements relative to lightly trafficked pavements. Table 2.4 outlines some possible distress limits for defining the terminal structural condition of pavements (Austroads 2003).

Table 2.4: Definition of terminal structural condition (Austroads 2003)

Road Function	Surface Deflection D_0, (mm)	Roughness Limit (IRI)	% Road Length with Rut Depth > 20 mm
Freeways, etc.	0.8	4.2	10
Highways and main roads (100 km/h)	0.85	4.2	10
Highways and main roads (80 km/h)	0.9	5.4	20
Other sealed local roads	1.6	No defined limit	No defined limit

2.4.1 Structural Number (SN)

The structural number (SN) (refer to Equation (2.5) and (2.6)) is the parameter that represents the pavement structural strength. It is given as the product of each layer thickness by its structural layer coefficient, which is an empirical coefficient representing each layer's relative contribution to the pavement strength (AASHTO 1993).

$$SN_i = a_i D_i \tag{2.5}$$

where,

SN_i = structural number of the i^{th} layer

a_i = layer coefficient of the i^{th} layer

D_i = thickness of the i^{th} layer, in inches

The overall load carrying capacity of a given pavement structure is then determined from the following:

$$SN = \sum_{i}^{n-layer} a_i D_i \tag{2.6}$$

Where, n-layer is the number of pavement layers above the subgrade.

SN has the advantage that it is related to the change in cumulative traffic loading and functional condition of the pavement (AASHTO 1993).

2.4.2 Modified Structural Number (SNC)

The modified structural number (SNC) is defined as the sum of the pavement structural number (SN) and the subgrade contribution (SN_{sg}) which is estimated from the Californian Bearing Ratio (CBR) of the subgrade (Hodges et al. 1975). Pavement's structural capacity may be quantified by using the SNC. The Road test estimation of pavement strength was further refined by the introduction of SNC (Watanatada et al. 1987), which directly took into account the subgrade contribution to pavement strength (Hodges et al. 1975). This method builds upon AASHTO's structural number concept, in which pavement layers are assigned a structural layer coefficient that is meant to represent the layer's contribution to pavement performance (ADB 1995).

The combined effects of drainage and rainfall on the behaviour of the pavement are thus represented through the effects of moisture on the strength properties and stiffness (e.g. resilient modulus) of the materials in each pavement layer. An increase in the moisture content of a material above the optimum associated with its density causes a decrease in the shear strength, and often a decrease in the stiffness, of the material. Thus it usually causes a decrease of the modified structural number and increase of the deflection (Watanatada et al. 1987).

In the AASHTO design procedure the subgrade's contribution to performance is considered through its resilient modulus, while in the Highway Design and Maintenance standard Model (HDM) (Watanatada et al. 1987), the subgrade contribution is incorporated in the SNC by considering its contribution to the SN. Although Paterson (1987) stated that SNC is derived from pavement layers up to a total thickness of 700 mm (suggesting that layers beyond a depth of 700 mm are not counted), the range of SNC in the Brazil-UNDP study goes up to nearly 8, while the maximum asphalt thickness was only 100 mm. This suggests that the contribution to the SNC of layers below 700 mm was, in some cases, included in the SNC estimation. In light of this anomaly, and in the absence of additional research to support an alternative, it is recommended that engineering judgment be applied to the calculation of an SNC when the structure exceeds 700 mm of thickness. While there are other methods of characterising pavement strength, the SNC is particularly useful because it is a single measure that is readily estimated from commonly available data. In that sense, it is more appropriate than deflection-based methods, for example, which require extensive testing and data manipulation to provide useful results (ADB 1995).

2.4.3 California Bearing Ratio (CBR) Testing

The California Bearing Ratio (CBR) Test is a penetration test developed by the California State Highway Department, USA to evaluate the soil strength and bearing capacity of sub grade soil for design of flexible pavement. Tests are carried out on natural or compacted soils in water soaked or un-soaked conditions and the results obtained are compared with the curves of standard test to have an idea of the soil strength of the sub grade soil. This value is broadly used and applied in design of the base and the sub-base material for pavement (Joseph and Vipulanandan 2011).

The CBR is a comparative measure of the strength of a nonstabilised material, including the subgrade and granular bases and subbases. It is the percentage of the resistance to penetration of the material of interest, by a standard piston at a standard rate, to that of a compacted crushed stone. CBR tests typically specify that the material of interest be tested after a 96-hour (4 days) soak, meant to be representative of the structure's lowest strength (which occurs when the pavement structure is saturated).

Ampadu (2007) examined the laboratory CBR test results on a subgrade material at different water contents for three different dry densities, and concluded that the rate of change in CBR per percentage change in water content during drying from the optimum Moisture Content (OMC) was 3 to 7 times larger than during wetting from OMC. Soaking from the OMC condition, leads to a relative reduction in CBR of between 46% and 98% for dry densities ranging between 1.71 and 1.36 Mg/m3. A linear log-log relationship between CBR and matric suction is suggested for matric suction values of up to about 15,000 kPa.

2.4.4 Resilient Modulus

Subgrade soil stiffness is an important parameter in pavement design. In recent years, mechanistic-empirical design procedures have attracted the attention of both pavement engineers and researchers. These design procedures require knowledge of the mechanical properties of the materials that make up the pavement structure (George 2003). The Resilient Modulus (M_R) is a measure of subgrade material stiffness (George 2003; Li and Selig 1994). The resilient response of unbound subgrade and granular subbase/base materials were highly nonlinear. It has become a well-known parameter to characterise unbound pavement materials because a large amount of evidence has shown that the elastic (resilient) pavement deflection possesses a better correlation to field performance than the total pavement deflection (Witczak et al. 1995).

AASHTO (1993) recommends using M_R as an input parameter to evaluate subgrade support. A material's resilient modulus is actually an estimate of its modulus of elasticity (E). While the modulus of elasticity is stress divided by strain for a slowly applied load, resilient modulus is stress divided by strain for rapidly applied loads - like those experienced by pavements (Li and Selig 1994). Resilient modulus is determined using the triaxial test. The test applies a repeated axial cyclic stress of fixed magnitude, load duration and cycle duration to a cylindrical test specimen. While the specimen is subjected to this dynamic cyclic stress, it is also subjected to a static confining stress provided by a triaxial pressure chamber. It is essentially a cyclic version of a triaxial compression test; the cyclic load application is thought to more accurately simulate actual traffic loading (Li and Selig 1994).

Resilient modulus, M_R (refer to Equation (2.7)) is usually determined by repeated load triaxial tests with constant confining pressure, σ_3, and with the deviator stress cycled between the hydrostatic state and some positive deviator stress $(\sigma_1 - \sigma_3)$ (Li and Selig 1994). For these conditions, the term resilient modulus is defined as follows:

$$M_R = \sigma_d / \varepsilon_r \tag{2.7}$$

where,

M_R	=	resilient modulus
σ_d	=	repeated deviator stress, $(\sigma_1 - \sigma_3)$
ε_r	=	recoverable (i.e. resilient) strain in the direction of axial stress σ_1 (major principal stress) with confining stress σ_3 (minor principal stress) constant

In recent years, determining the influence of moisture changes on resilient modulus (M_R) of subgrade soils beneath a pavement has increased as AASHTO (1993) recommended using a single M_R value to account for the seasonal variation in subgrade moisture content, known as the effective roadbed M_R (Khoury and Zaman 2004).

2.4.5 Dynamic Cone Penetration Test

In road construction, there is a need to assess the adequacy of a subgrade to behave satisfactorily beneath a pavement. Proper pavement performance requires a satisfactorily performing subgrade (Salgado and Yoon 2003). The dynamic cone penetration (DCP) test was developed in Australia by Scala (1956). The DCP test has proven to be a valuable tool for the rapid measurement of existing pavement strength properties, including the CBR. DCP testing can be applied to the characterization of subgrade and base material properties in many ways. Perhaps the greatest strength of the DCP device lies in its ability to provide a continuous record of relative soil strength with depth. One of the major applications of DCP testing has been in the structural evaluation of' existing pavements (Burnham and Johnson 1993).

Brisbane City Council (BCC) has developed an empirical correlation between DCP penetration and subgrade CBR specifically for use within the Brisbane area. The correlation is shown in the following Equation (BCC 2011),

$$\mathrm{CBR}_{\mathrm{Subgrade}} = 83.048(\mathrm{DCP})^{-0.7191} \tag{2.8}$$

where

$\mathrm{CBR}_{\mathrm{Subgrade}}$ = Subgrade CBR value (%)

DCP = DCP Penetration (mm/blow)

2.5 ESTIMATING STRUCTURAL NUMBER FROM FWD DEFLECTIONS

This section reviews a number of available methods of calculating structural number, to select the best method for analysis in this study. A number of methods are available to estimate the modified structural number (SNC) and adjusted structural number (SNP) from deflection data (Morosiuk et al. 2001). The methods outlined below are those that are considered appropriate for Australian conditions (Austroads 2003).

2.5.1 Estimating Modified Structural Number from FWD Deflections

Calculation of the SNC requires some material properties (thickness and layer coefficients), and there are cases where it may be desirable to estimate the SNC based on deflections instead. Such relationships exist for several commonly available deflection measuring devices. Paterson (1987) provided relationships between Benkelman beam deflections and SNC for pavements on granular and cemented bases, based on Brazil data, and these are provided in Equations (2.9) and (2.10).

For unbound granular based pavements, Equation (2.9) and for cemented or bitumen stabilised pavements, Equation (2.10) are recommended (Paterson 1987):

$$\mathrm{SNC_i} = 3.2 \times \mathrm{D_o}^{-0.63} \tag{2.9}$$

$$\mathrm{SNC_i} = 2.2 \times \mathrm{D_o}^{-0.63} \tag{2.10}$$

where,

$\mathrm{SNC_i}$ = modified structural number at age ‘i’ (Paterson 1987)

$\mathrm{D_0}$ = maximum deflection (mm) at load centre at age ‘i’

SNC includes the contribution of the subgrade (SN_{sg}) and the strength coefficients of the material properties (a_1) under the in situ conditions of the local environment of the pavement.

For asphalt pavements:

$$SNC_i = 3.51 \times [\log_{10}(CBR)] - 0.85 \times [\log_{10}(CBR)]^2 + 0.26 + \frac{842.8}{D_0 - D_{1500}} - \frac{42.94}{D_{900}} \quad (2.11)$$

(from Hodges et al. (1975) and Jameson (1993) for asphalt pavements)

where,

$$\log_{10}(CBR) = 3.264 - 1.018 \times \log_{10}(D_{900}) \quad (2.12)$$

for asphalt pavements from Jameson (1993).

D_{900} and D_{1500} = maximum deflection in microns (adjusted to surface temperature) at age 'i', at 900 mm and 1500 mm from the load centre, respectively.

2.5.2 Jameson's Method

The structural number (SN) is calculated using the following relationship (refer to Equation (2.13)) derived from Jameson (1993) using data of a range of pavements (Austroads 2003):

$$SN = 1.69 + \frac{842.8}{D_0 - D_{1500}} - \frac{42.94}{D_{900}} \quad (2.13)$$

where,

D_0 = maximum deflection (mm) at load centre

D_{900} = deflection (mm) 900 mm from centre of FWD test plate

D_{1500} = deflection (mm) 1500 mm from centre of FWD test plate

The other terms in Equation (2.13) are as defined previously. The deflections are all normalised to a stress of 700 kPa.

The California Bearing Ratio (CBR) of the subgrade is determined using the following Equation (Jameson 1993):

$$\text{Log}_{10}(CBR) = 3.264 - 1.018 \times \text{Log}_{10}(D_{900}) \quad (2.14)$$

The CBR is then used to calculate the structural contribution of the subgrade, SN_{sg}, as shown below (Hodges et al. 1975):

$$SN_{sg} = 3.51 \times \mathrm{Log}_{10}(\mathrm{CBR}) - 0.85 \times \left(\mathrm{Log}_{10}(\mathrm{CBR})\right)^2 - 1.43 \quad (2.15)$$

The adjusted structural number, SNP, is determined from the sum of the structural number, SN, and the contribution of the subgrade, $SN_{sg,}$ as shown below (Hodges et al. 1975), assuming that the modified structural number, SNC, is approximately the same as SNP for most Australian pavements:

$$\mathrm{SNC/SNP} = \mathrm{SN} + SN_{sg} \quad (2.16)$$

2.5.3 Robert's Method

Roberts (1995) developed the following relationship for the structural number, SN, based on FWD data collected in Australia and the Philippines (Austroads 2003):

$$\mathrm{SN} = 12.992 - 4.167 \times \mathrm{Log}_{10}(D_0) + 0.936 \times \mathrm{Log}_{10}(D_{900}) \quad (2.17)$$

with the structural contribution of the subgrade estimated using Equations (2.14) and (2.15). This allows the adjusted structural number, SNP/SNC, to be estimated from Equation (2.16).

(Loizos et al. (2002)) derived the following relationship to estimate the SNP for asphalt pavements in Greece using FWD deflections:

$$\mathrm{SNP} = 167 \times (D_0) - 0.57 \quad (2.18)$$

2.5.4 Salt's Method

Salt and Stevens (2001) developed the following relationship for the adjusted structural number, SNP, based on FWD deflections on sealed granular pavements in New Zealand:

$$\mathrm{SNP} = 112 \times (D_0)^{-0.5} + 47 \times (D_0 - D_{900})^{-0.5} + 56 \times (D_0 - D_{1500})^{-0.5} - 0.4 \quad (2.19)$$

where, D_0, D_{900} and D_{1500} are as defined above, except that the test plate pressure is normalised to a stress of 566 kPa. This means that if the test plate pressure is 700 kPa then the deflections are adjusted by a ratio of 566/700 (0.809).

2.5.5 Comparison of SNP and SNC estimation for granular pavements

A comparison was undertaken between the Adjusted Structural Number, SNP, estimated by Jameson (1993), Roberts (1995) and Salt and Stevens (2001) using deflections D_0, D_{900} and D_{1500}, and the Modified Structural Number, SNC, estimated by Paterson (1987) using the deflection D_0. The comparison was based on a sealed granular pavement network with 688 FWD tests. The results showed that all the relationships between SNC and SNP are closely related for this network data set, particularly the Salt and Stevens (2001) estimate of SNP. This outcome showed that the additional bowl deflection terms D_{900} and D_{1500} do not provide a significantly different estimate of SNP from that based on only the D_0 deflection. The above data set appears to be reasonably representative of a sealed granular pavement network as it has a wide range of SNC and SNP values. The correlation of the Salt and Stevens (2001) estimate of SNP with the Paterson (1987) estimate of SNC is remarkable. This is particularly so because the Paterson (1987) estimate of SNC was based on Benkelman Beam deflections on Brazilian pavements while Salt and Stevens (2001) used FWD deflections on New Zealand pavements to estimate SNP (Austroads 2005).

Therefore, comparisons of the various means of estimating SNC and SNP using either the maximum bowl deflection, D_0, or a range of bowl deflections ($D_{0,}$ $D_{900,}$ and D_{1500}), suggest that network level assessment of SNC or SNP could be based on the D_0 deflection without any significant loss in accuracy (Austroads 2003; Martin and Crank 2001). The bowl deflections other than, $D_{0,}$ do not significantly improve the estimation of the SNC with the strength and deflection relationships. This implies that a network level assessment of pavement strength only needs measurement of the maximum deflection, D_0 (Austroads 2003; Martin and Crank 2001). To avoid confusion, only modified structural number, SNC is used throughout this study. Equations developed by Paterson (1987) are used in this research for the estimation of the SNC.

2.6 EVALUATION OF FUNCTIONAL PERFORMANCE OF PAVEMENTS

An essential component of any PMS is condition prediction. Before pavement maintenance strategies and repair budgets can be prepared or evaluated effectively, pavement managers must assess the current and future condition of their pavement network. Accuracy in prediction is essential for budget forecasting and work planning (Shahin et al. 1994).

The functional performance of a pavement concerns how well the pavement serves the user. In this context, riding comfort or ride quality is the dominant characteristic. The "serviceability-performance" concept was used to quantify riding comfort (AASHTO

1993). The serviceability-performance concept is based on five fundamental assumptions, summarised as follows:

1. Highways are for the comfort and convenience of the traveling public (User).
2. Comfort, or riding quality, is a matter of subjective response or the opinion of the User.
3. Serviceability can be expressed by the mean of the ratings given by all highway users and is termed the serviceability rating.
4. There are physical characteristics of a pavement which can be measured objectively and which can be related to subjective evaluations.
5. This procedure produces an objective serviceability index. Performance can be represented by the serviceability history of a pavement.

The serviceability of a pavement is expressed in terms of the present serviceability index (PSI). The PSI is obtained from measurements of roughness and distress, e.g. cracking, patching and rut depth (flexible), at a particular time during the service life of the pavement. Roughness is the dominant factor in estimating the PSI of a pavement (AASHTO 1993).

Table 2.5 shows the following tentative recommendations, made as a practical guide to discrete network level sampling of pavement lengths for given road types or functional road classifications (Austroads 2003).

Table 2.5: Tentative recommendations for performance indicators to undertake discrete network level sampling (Austroads 2003)

Road Type	**Roughness**		**Rutting**		**Cracking**	
	Limit (IRI)	**Rate (IRI/yr)**	**Limit (mm)**	**Rate (mm/yr)**	**Limit (% area)**	**Rate (% area/yr)**
Freeway	3.5	0.05	10	0.3	1	0.1
Highly trafficked arterial road (AADT > 10,000)	4.2	0.08	10	0.5	2	0.1
Medium trafficked arterial road (AADT 10,000 – 2,000)	4.2	0.2	15	0.6	5	0.5
Low trafficked arterial or main road (AADT < 2000)	5.4	0.3	20	0.8	10	1
Note: Sampling to occur when either or of any of the above performance indicator limits and/or rates is exceeded.						

2.6.1 Roughness

Roughness is a condition parameter that characterises deviations from the intended longitudinal profile of a pavement (Austroads 2007). It is used as a measure of the rideability of the road surface (Foley 1999) and can be an indicator of the serviceability and/or structural condition of a pavement. It is influenced by surface irregularities, distortions and deformations. The roughness of a pavement usually increases with time from initial construction to ultimate retirement. It is generally assumed that as roughness increases the structural condition of the pavement decreases. In addition, as roughness increases so too does the dynamic pavement loading (TMR 2012). Measurement of roughness focuses on characteristic dimensions that affect vehicle dynamics and hence road user costs, ride quality and dynamic pavement loads (Austroads 2007). Roughness can develop from loading of the pavement and from other factors (e.g. from material volume changes associated with moisture changes). Identifying the causes of roughness can be critical with respect to selecting an appropriate rehabilitation treatment (TMR 2012).

Road roughness is measured in terms of the International Roughness Index (commonly referred to as IRI) for the lane (Austroads 2007). The IRI summarizes the longitudinal surface profile of the wheel path and it is computed from the surface elevation data collected by either a level survey or a multi-laser profilometer (MLP). Usually, MLP is used to measure the longitudinal profile in each wheel path with lasers. The IRI is defined by the average rectified slope (ARS), which is the ratio of the accumulated suspension motion to the distance travelled and obtained from a mathematical model of a standard quarter car traversing a measured profile at a speed of 80 km/h. It is expressed in units of meter per kilometre (m/km) (Meegoda and Gao 2014).

Laser profilers provide data to calculate IRI (m/km) from measured road profile or provide an estimate of NAASRA (National Association of Australian State Road Authorities). It should be noted that it has been replaced by Austroads as roughness counts. NAASRA Counts is based on the NAASRA standard passenger vehicle used to collect roughness data. This road-response-type meter measures units of 15.2 mm of vertical movement of the vehicle body relative to its rear axle as one NAASRA count. Roughness is expressed in NAASRA counts per kilometre (c/km).

A reasonable correlation between the two roughness indices is provided by the following relationship (Prem 1989):

$$\text{NAASRA counts/km} = 26.5 \times (\text{lane - IRI}) - 1.27 \tag{2.20}$$

Results are normally processed and reported at 100 m intervals for each lane surveyed (Austroads 2011).

The progression of roughness with time is a complex phenomenon (Paterson 1987). Paterson (1987) showed that composite distress depends on deformation due to traffic loading and rut depth variation, surface defects from cracking, potholes, and patching, and a combination of aging and environmental factors. Madanat et al. (2005) analysed the roughness data from Washington State's PMS database and showed that the most relevant predictors of change in IRI for AC pavements and overlays are previous year IRI value, cumulative number of equivalent single axle loads (ESALs), base thickness, total thickness of AC, including all overlays, age of pavement, minimum temperature in the coldest month (average over the life of the pavement), and annual precipitation (average over the life of the pavement). Perera and Kohn (2001) using long-term pavement performance (LTPP) data showed that design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress are major factors causing changes in flexible pavement smoothness (Meegoda and Gao 2014).

The initial step in evaluating the road roughness data is to compare the measured values against acceptable values set by the road agency, which may vary with the design speed and the function of the road. Roughness data can provide some insight into the distress of a pavement, especially when considered with other condition parameters and field observations.

Acceptable values of roughness are usually established by road authorities, and whilst they may vary between jurisdictions, some indicative levels are listed in Table 2.6. When considering roughness data for a given road the results should be compared against acceptable values, and judgement made as to whether further investigation is warranted. Time series of roughness data can provide insight into the likely future trend of roughness for road lengths, and allow estimations as to when investigatory and intervention levels may be reached in the future (Austroads 2007).

Table 2.6: Levels of roughness (Austroads 2007)

Road function	**Typical maximum desirable roughness(IRI, m/km) for new construction or rehabilitation (length 500 m)**	**Indicative investigation levels for roughness (IRI m/km)**	
		Isolated areas	**Length > 500 m**
Freeways and other high-class facilities	1.6	4.2	3.5
Highways and main roads (100 km/h)	1.9	5.3[a]	4.2
Highways and main roads (< 80 km/h)	1.9	6.1	5.3
Other local sealed roads	No limits defined [b]	No limits defined [b]	No limits defined [b]
Notes: a. Lower values may be appropriate where total traffic or heavy vehicle volumes are high.			
b. Roughness levels depend on local conditions and traffic calming measures.			

2.6.2 Rutting

Rutting is a longitudinal deformation (depression) located in wheel paths and is commonly found in flexible pavements. Generally, the layer(s) suffering the deformation will be evident from associated indicators, or may be determined by inspection of test pits or trenches that reveal the pavement (cross) section through (across) the rut(s) (TMR 2012). One of the criterion used as part of the pavement design procedure is to limit rutting of the subgrade, or the cumulative permanent deformation caused by vertical compressive strain at the top of the subgrade layer, to a certain level. Subgrade rutting may indicate that:

- the pavement is performing in accordance with the (original) design assumptions;
- the (original) design traffic has been exceeded;
- the effective subgrade strength is/was less than the design strength adopted in the (original) design; or
- the in situ condition of the subgrade is/was different from the design condition adopted in the (original) design (e.g. moisture content is higher); or
- the pavement has suffered from one or more overloads (e.g. an over mass vehicle traversing the pavement) (TMR 2012).

Permanent deformation of the subgrade accumulates with the passage of each (heavy vehicle) axle (group) (TMR 2012). Rutting may occur as a result of permanent deformation in granular bases, asphalt surfacings or bases, in the subgrade or in a combination of these. Where excess bitumen has been placed in a seal, or many seals have been placed over time resulting in a significant total thickness of seals, rutting may occur in the seal (TMR 2012).

Deformation of a granular base could be as a consequence of very high (volume and/or mass) traffic loading, poor material quality, excessive moisture or inadequate compaction/construction. Deformation of an asphalt base may be the result of very high traffic loading (e.g. > 10^7 ESAs), inappropriate mix design, inappropriate asphalt selection, interaction between layers (e.g. excess cutter in a priming seal penetrating an overlying asphalt layer) or inadequate compaction. The evaluation should consider, and focus materials testing on these aspects (TMR 2012).

In addition to its effect on serviceability, deformation in base layers may lead to a reduction in the effective pavement thickness and, if left untreated, to the premature development of deformation in the subgrade. This deformation may progress to shoving if the rutting becomes so severe that surface cracking occurs and moisture enters and weakens the underlying layers and/or the subgrade (TMR 2012).

To assist in identifying the cause of rutting, the existence or absence of associated shoving is an important attribute. For a flexible granular pavement, where ruts are wide and there is little or no evidence of shoving, it is more likely related to deformation at depth in the pavement (e.g. at the subgrade level) as a result of either insufficient pavement strength and/or compaction of the pavement under traffic. In this case inadequate pavement strength is the result of pavement layers being too thin or of insufficient quality to distribute the applied load sufficiently to avoid overstressing lower layers in the pavement or the subgrade (TMR 2012).

To assess whether rutting is due to inadequate pavement strength it is useful to plot measured pavement deflections at various chainages against measured rut depths at the time of deflection testing. The higher the correlation of rut depth and deflection the more likely the rutting is at least partly due to inadequate pavement strength. If rut depths do not correlate with pavement deflection and there is little or no shoving, the most likely cause is densification of the pavement layers under traffic early in the life of the pavement (TMR 2012).

Where rutting is associated with shoving, it is usually indicative of the shear strength in the upper pavement layers being inadequate to withstand the applied traffic loads. In this case, there will be poor correlation between the severity of rut depth and measured deflections. Trenching across the full width of the lane(s) and/or asphalt coring within and between ruts can also be used to identify the critical pavement layer or layers. Laboratory testing of the affected materials sampled from the pavement will further assist in evaluating whether the shear deformation is related to deficiencies in the specification or standard for that material, deficiencies in construction or to the use of non-conforming material (TMR 2012).

Shoving represents a gross deformation of the pavement that rapidly leads to disintegration. Therefore, it cannot be tolerated. It generally occurs because of one or more of the following:

- Inadequate strength in surfacing or base material.
- Poor bond between pavement layers.
- Lack of containment of the pavement edge.
- Inadequate pavement thickness overstressing the subgrade (TMR 2012).

Rutting is measured (TMR 2012):

- as the maximum vertical displacement in the transverse profile (i.e. perpendicular to flow of traffic);
- across one wheel path or both wheel paths within a traffic lane; and
- relative to a reference datum.

It can be measured in each direction. Various measures are available including (TMR 2012):

- Deviations from a 1.2 m long straight edge (e.g. measured during a visual assessment);
- The recording of the transverse profile of the road surface using a “rut meter”;
- Rutting calculated using the results of a laser profilometer survey. The 1.2 m straight edge or equivalent results are used by TMR.

Automated methods are being used increasingly. However, manual measurements can still have a place in project level investigations. For example manual measurements can be used to validate automated measurements, to check particular locations (spot checks) or to compare current rut depths against those recorded in the last automated survey (which may be some time ago) (TMR 2012).

The results can be processed and reported for road segments of a specific length (e.g. average rut depth over section length, percentage of section with a rut depth > 10 mm). Usually state wide roughness surveys are conducted annually. One of the criteria used as part of the pavement design procedure is to limit rutting of the subgrade, or the cumulative permanent deformation caused by vertical compressive strain at the top of the subgrade layer, to a certain level (TMR 2012).

2.7 FACTORS AFFECTING DETERIORATION OF PAVEMENTS

As the pavement maintenance planning depends on the ability of the road authority to accurately predict future pavement condition, it is very important to be able to specify and estimate an accurate model to predict deterioration. Failure to do so will result in incorrect choices of maintenance strategies and consequently inefficient utilisation of resources. A deterioration prediction model should include all the factors that affect deterioration of the pavement as completely as possible. It should accurately represent the effect of maintenance on pavement condition (Ramaswamy and Ben-Akiva 1990). The main focuses of this research is to modelling deterioration of Asphalt Concrete (AC) pavement. Hence, factors affecting deterioration of AC pavement are reviewed in this section. The major factors influencing the loss of serviceability of a pavement are traffic, age, and environment (AASHTO 1993). The factors that affect the deterioration of pavement can be categorised as follows, after Ramaswamy and Ben-Akiva (1990):

- Pavement characteristics: pavement strength, layer thicknesses, base type, surface type.
- Pavement history: time since last rehabilitation, total pavement age.
- Traffic Characteristics: average daily traffic, cumulative traffic, traffic mix (percentage of trucks).
- Environmental variables: average monthly precipitation, number of freeze-thaw cycles, average annual minimum temperature.

Pavement infrastructure deterioration is an aggregated impact from traffic loading, environmental condition, and other contributors. The behaviour of a pavement under these factors depends on the characteristics of its structure (materials and thickness of each pavement layer), the quality of its construction, and the subgrade (bearing capacity and presence of water) (Schaefer et al. 2008). Each factor causes certain distresses on the pavement. Understanding factors that lead to deterioration of roads help infrastructure managers to refine their construction and maintenance specifications (Shamsabadi et al. 2014).

Cracking and rutting caused by pavement bending under traffic loads are two of the most prominent forms of distresses. Tyre pressure produced by vehicles in the radius of loaded area induces tensile stress on the pavement, lateral shear in the surface and vertical stress at the subgrade which gradually deteriorate the pavement (Shamsabadi et al. 2014). Severity of distresses and the pace of their formation are heavily influenced by material properties of the pavement. Strength and bearing capacity, gradation, modules of elasticity and resilience of the materials used in construction determine pavements' endurance under load and climate fluctuations (Shamsabadi et al. 2014).

On the other hand, construction quality influences the two significant factors in initiation of top-down cracking: voids and aggregate gradations caused. Construction quality also determines the initial pavement condition which has an impact on the pace that pavement failures occur. Environmental conditions such as climate oscillations, precipitation and freeze/thaw cycles are the primary causes of some dominant distresses such as longitudinal and transversal cracks. Temperature fluctuations are followed by tensile and compressive stress in pavement which initiates thermal cracking (Shamsabadi et al. 2014).

Studies on pavement performance evaluations show that other than formation of longitudinal and alligator cracks, roughness of the road also worsens with a boost in precipitation. In cold regions, water penetrating into the pavement layers freezes in the winter. Thaw of these ice particles during spring causes deformation in pavement layers and triggers fatigue cracking (Shamsabadi et al. 2014).

Each of the factors has different impacts on pavement deterioration. Pavement engineers need to understand and identify the most influential factor that escalates pavement deterioration following an extreme weather event such as flooding. Regular monitoring and non-destructive testing are necessary to identify the causes of escalated deterioration of flood affected pavements.

Interactions between climate, vehicles and the road result in deformation and deterioration of pavements - predicting this behaviour is not easy. While deterioration models for rigid pavements perform relatively well, because of the high visco-elastic characteristic of the asphalt, current deterioration models for flexible pavements have had limited success to date (Shamsabadi et al. 2014).

2.8 PAVEMENT DETERIORATION MODELS

Deterioration modelling is a key component in a transportation infrastructure management system. It serves to capture and predict the performance of a facility. A sound deterioration model should incorporate: (1) relevant variables that affect deterioration processes; (2) physical principles that reflect deterioration mechanisms; and (3) rigorous statistical approaches for estimating the model (Hong and Prozzi 2010). Both deterioration and policy optimisation models play a key role in pavement infrastructure management. A successful management system requires sound deterioration modelling as the basis for the policy optimisation modelling and an effective integration of these two components (Hong and Prozzi 2014).

Pavement deterioration models are not only important for road agencies to manage their road network, but also in road pricing and regulation studies. Both the deterioration of the pavement over time and the relative contribution of the various factors to deterioration are important inputs into such studies. Useful models should be able to quantify the contribution of the most relevant variables to pavement deterioration. Some of these variables are the pavement structure (materials and strength), traffic (axle configuration and axle loads) and environment conditions (temperature and moisture) (Prozzi and Madanat 2004).

A PMS consists of three critical elements: (1) a performance database, which contains historical pavement condition information; (2) a series of deterioration models, which predict pavement performance; and (3) optimisation models, which determine optimal policy in maintenance and rehabilitation decision making (M&R) work (Hong and Prozzi 2014).

With the development of and easy access to automatic survey equipment, such as high-speed inertia profilers, laser rut bars, and other equipment able to collect pavement condition information at highway speeds, many road agencies conduct annual pavement condition surveys and have well-established databases to support the management of their highway network. In comparison, the development of accurate performance prediction models and maintenance and rehabilitation policy optimisation routines are relatively less emphasised by most road agencies. To make the best of the valuable data collected, sound modelling for both pavement deterioration and policy optimisation is warranted. Accurate and comprehensive data with unrealistic and simplistic modelling are self-defeating. In recent years, continuous effort has been made toward more advanced deterioration modelling in the area of pavement management (Hong and Prozzi 2014).

Prediction of pavement deterioration and subsequently, the time of periodic maintenance and rehabilitation works on a road infrastructure facility would be achieved through identification of the variables. The variables contribute to the pavement deterioration and the development of a model that expresses their contributions. Current concepts of pavement performance include some consideration of functional performance, structural performance, and safety (AASHTO 1993). However, some of the prediction models still consider evaluation of pavement performance as a study of the fundamental behaviour of a section or length of a pavement in terms of its historical riding quality and traffic (Haas et al. 1994; Jackson et al. 1996). Moreover, there are also complex models that require a large systematic database with numerous environmental, traffic load related and material properties variables. Where the data are available, there is often a problem of insufficient data recording (Lukanen and Han 1994). Worse still, the accuracy of such complex models has been found to be very low (Shahin et al. 1987). All these problems make it necessary for a road agency to have a simple model that makes use of the available data without having to compromise the accuracy of the final prediction results (Maina et al. 1999).

With the assistance of a good and effective prediction model, decision makers would be able to perform proper financial planning and life-cycle economic analyses to determine the timing and prioritisation of the methods and types of maintenance and rehabilitation works on a particular road section. To emphasise the importance of timely and effective maintenance and rehabilitation programs, numerous cost analysis studies on pavement maintenance and rehabilitation have shown that total annual maintenance costs for pavements that are maintained perpetually in a good to excellent condition are 4 to 5 times lower than that for pavements that are allowed to cycle through to poor and failed condition and then repaired (Maina et al. 1999; Saraswatula and Amirkhanian 1992).

2.9 PAVEMENT DETERIORATION MODELS FOR FLOOD AFFECTED ROADS

Local government authorities across Australia have adopted a variety of PMSs for local roads to guide the development of pavement maintenance, rehabilitation programmes, and appropriately allocate road funding. Many practitioners managing local roads (the majority of which are considered low volume) are becoming disenchanted with their PMS packages, since the forecast of pavement performance does not accurately reflect actual pavement condition (Giummara et al. 2007).

The main reason for the poor correlation between predicted PMS outputs and existing road conditions was that the pavement prediction models used in existing PMSs were based on the performance of the overseas pavements (typically the World Bank's road deterioration models in HDM-III [The Highway Design and Maintenance Standards Model]). None of these approaches reliably predicted pavement performance under Australian conditions (Martin 1996). New models have been derived or existing models have been modified to predict the pavement performance or deterioration under Australian climatic conditions (Martin 1996). However, this task is an ongoing process.

Recent flood and cyclone events present new challenges to efficiently maintain road pavements. It raises the need for monitoring road pavements in flood prone areas which is the main key to understand how roads deteriorate under extreme weather conditions such as flooding. Modelling the rapid deterioration of such pavements is also important to avoid long term consequences of not including flooding impacts in the PMS. Long-term observation of pavements is also instrumental in providing answers to why some roads survived flooding but others were greatly impacted.

Modelling performance of pavements subjected to flooding is not a simple task. Unlike the conventional approach of modelling pavement deterioration under normal climatic conditions, flooded pavements introduce a new dimension into the already complex mathematical model when pavements are either partially or fully saturated. These dynamic conditions add new challenges to efficiently manage the road asset. Moreover, condition data prior to and after flooding events needs to be gathered. This data will provide valuable information for building resilience into future road pavements and predicting the cost consequences when resilience cannot be built-in.

Although the purpose of the road deterioration modelling is to predict the pavement condition, the increasing frequency of extreme weather events is adding new uncertainties in the modelling as the road pavement designs are based on the past history of the location. Structural strength of pavements can be influenced by the increasing frequency of extreme rainfall events causing flooding. It affects the rate of pavement deterioration. Road pavements are designed based on moisture and temperature patterns reflecting the historical local climate of the location. Roads are now subjected to different climatic conditions over the design life than was originally expected.

The prediction of pavement performance is a critical element of the pavement management systems currently being developed in Australia (Austroads 2010a). The effects of extreme climate on pavement deterioration can have significant impacts on the planning and management for both maintenance and rehabilitation of the road pavements. For example, Chai et al. (2010) indicated that environmental and climate factors had the greatest effect on the pavement performance of the local roads in Southeast Queensland (SEQ). Although the field experiments were initially conducted to develop the HDM-III predictive models to cover a wide range of conditions, there remain local factors that cannot be introduced into the model. Calibration of the HDM model to local conditions is always necessary and recommended. Moreover, if calibration is not carried out, the actual pavement deterioration trends and the predicted deterioration may be very different. The lack of an appropriate local calibration can result in an underestimation, or overestimation, of the budget allocation of highway expenditure (Chai et al. 2010).

To summarise, development of new models specifically for flood affected roads are necessary to improve the decision making process of a PMS and better use of rehabilitation funds after flooding.

2.10 GENERAL APPROACH TO PREDICT PAVEMENT PERFORMANCE

Pavement performance prediction models are often regarded as the most important component of a pavement management system (Martin 1996). Modelling pavement performance is necessary to a PMS at both project level and national network level.

Project level models are different from and more detailed than network level models as they are used in the analysis and design of pavements, life-cycle cost analyses of alternative designs, and other related purposes. At the project level, pavement performance is defined by the distress, loss of serviceability index and skid resistance, loss of overall condition, and by the damage that is done by the expected traffic (Lytton 1987). At the **district network level**, performance is defined not only by the condition and trends of individual projects but also with the overall condition of the network and with the level of performance that is provided by each type and functional class of road (Lytton 1987).

Network level models are necessarily less detailed but are used in the selection of optimal maintenance and rehabilitation strategies, size and weight and cost allocation studies, and network level trade-off analyses between pavement damage, maintenance, and user costs. Development of a pavement deterioration model has to be based on statistics to allow the experimental testing of data and mechanics to satisfy the technical and economic requirement of managing pavements (Lytton 1987). At **state or province level**, there is less concern with the conditions and trends of individual projects but with measures of the overall condition of the pavement networks in each geographical subdivision, especially as they reflect the needs for present and future funding and the effects on user costs (Lytton 1987).

Modelling pavement performance or deterioration approaches are grouped into two classes: deterministic and probabilistic (Lytton 1987).

2.10.1 Deterministic Model

The **deterministic model** is a mathematical function that predicts the future pavement condition as a precise value (Abaza 2004). **Deterministic approaches** predict a single value of the dependent variable from pavement performance prediction models based on statistical relationships between the dependent and independent pavement performance variables. Deterministic models of pavement performance are relationships composed of the variables understood or assumed to influence pavement performance. The resulting predictions take no direct account of the stochastic nature (elements of randomness) of pavement performance (Martin 1996). This approach includes primary response, structural performance, functional performance and damage models (George et al. 1989). Deterministic model types are defined on the basis of their derivation (Martin 1996). These model types are classified as follows:

- Mechanistic models
- Mechanistic-empirical models

- Empirical models

Mechanistic models are based on a fundamental and primary response approach to predicting pavement performance, such as elastic theory (Lytton 1987; Martin 1996). Mechanistic models are based on a physical representation of the pavement deterioration process. However, due to the complexity of the road deterioration process, this approach is, at present, unfeasible (Prozzi 1999).

Mechanistic-empirical models are based on theoretical postulations about pavement performance, but are calibrated, using regression analyses, by observational data. These models must adhere to known boundary conditions and physical limits. These models can incorporate interactive forms of distress near the end of pavement life, such as the interaction of rutting with cracking, when these interactions are well understood. If these models are theoretically sound and correctly calibrated, they may be applied beyond the range of data from which they were developed (Lytton 1987). These models use material characterisation (usually laboratory testing) and pavement response models (usually linear elastic or finite element type models) to determine pavement response. This constitutes the mechanistic component. The calculated pavement response (critical strain, stress or deflection) is correlated with pavement performance and finally calibrated to an actual pavement structure. Pavement test sections are used for this purpose as well as in-service pavement sections. Empirical and mechanistic-empirical models are currently the most widely used models despite their limitations. Empirical models based on regression analysis have been used for many years and constitute some of most widely used deterioration models. However over the past 20 years there has been a tendency for road agencies to direct their efforts towards mechanistic-empirical models because of the appeal from an engineering point of view (Prozzi 1999).

The main advantage, which mechanistic-empirical models claim, is their ability to extrapolate predictions out of the data range and conditions under which they were calibrated, thus producing deterministic performance predictions. This advantage constitutes, in turn, their main disadvantage since it is impossible to assess the reliability of the predictions when these models are used out of the data range for which they have been calibrated (Prozzi 1999).

Empirical models are developed from regression analyses of experimental or observed data. These models are useful when the mechanism of pavement performance is not understood, however, they should not be used beyond the range of data from which the model was developed (Lytton 1987). An empirical design approach is based solely on the results of experiments or experience. Observations are used to establish correlations between the inputs and the outcomes of a process, e.g. pavement design and performance. These relationships generally do not have a firm scientific basis, although they must meet the tests of engineering reasonableness (e.g. trends in the correct directions, correct behaviour for limiting cases, etc.). Empirical approaches are often used as an expedient when it is too difficult to theoretically define the precise cause-and-effect relationships of a phenomenon (Schwartz and Carvalho 2007).

In Empirical models, the dependent variable is any pavement performance indicator of interest. Both aggregate indicators of performance [such as the Present Serviceability Index (PSI), the Riding Comfort Index (RCI), or the Pavement Condition Index (PCI)] and individual performance indicators (such as skid resistance, rutting or cracking) have been used as dependent variables. These dependent variables are related to one or more explanatory variables representing pavement structural strength, traffic loading, and environmental conditions (Prozzi 1999).

In some of these models, explanatory variables are used and discarded solely based on considerations of availability and the statistics of their parameters. Often, relevant variables are discarded due to low statistical significance (usually based on the t-statistic of the corresponding parameter) of their parameters. On the other hand, irrelevant variables are often incorporated into the model based on the same considerations (Prozzi 1999).

Most of the specifications available in the literature are just linear combinations of the available variables. The criterion typically used to select the best specification from among several alternatives is to obtain the best possible fit to the data (usually measured by the coefficient of determination, R^2). In the better empirical models, the specification forms are based on physical laws, or at least, they intend to simulate the actual physical process of deterioration. The specification, even when relatively simple (as compared with the actual physical phenomenon), is not constrained to linear equations. Also, relevant variables, whose parameters are not statistically significant for the given sample, remain in the specification of their t-statistics (Prozzi 1999).

A well-developed performance model combines both statistics (experimental design) and mechanics (Lytton 1987). Application of effective techniques in modelling flexible pavement rehabilitation and management requires pavement performance condition feedback. The required performance feedback is typically obtained from field measurements of pavement distress conducted using appropriate inspection procedures. The field measurements are usually performed annually or biennially on pavement systems with similar material properties and loading conditions. The collected pavement distress data is then used to study the performance of pavements over time, and to predict the future performance of similar pavements. Prediction of future pavement condition is an essential factor for effective application of any pavement rehabilitation and management model (Abaza 2004).

2.10.2 Probabilistic Model

The **probabilistic model** is a probability function that predicts the future pavement condition with a certain level of probability. Probability levels are assigned to possible future condition outcomes based on engineering judgment or from an analysis of past performance of pavements. **Probabilistic approaches** inherently recognise the stochastic nature of pavement performance by predicting the distribution of the dependent variable (Martin 1996). They include survivor curves, Markov and semi-Markov transition processes (Lytton 1987).

2.11 HDM4 MODEL

The roughness model of the World Road Association's Highway Development and Management (HDM-4) software program (Kerali et al. 1998) consists of the predictions for each component of roughness (cracking, disintegration, deformation, and maintenance). The total incremental roughness is the sum of these components and the environmental component. The last term of the roughness equation is the environmental component and it is due to factors which include temperature and moisture fluctuations, and also foundation movements. The structural and the environmental components of the roughness deterioration model are related to the environmental coefficient (m) and the calibration factor (K_{gm}) as shown in Equation (2.21). The expression for roughness progression is as follows:

$$\Delta RI = K_{gp}[134\exp(m \times K_{gm} \times AGE3)\ (1 + SNPK_b)^{-5} YE4] + [0.0066 \times \Delta ACRA] + [0.088 \times \Delta RDS] + [0.000199(2 - FM)\ \{((\Delta NPT_a \times TLF/2) + (\Delta NPT \times TLF/2))^{1.5}\ (NPT_a)^{1.5}\} \times c] + [m \times K_{gm} \times RI_a] \quad (2.21)$$

where,

ΔRI	=	incremental change in international roughness index during the analysis year (IRI m/km)
$SNPK_b$	=	adjusted structural number of pavement for cracking at the end of the analysis year
$\Delta ACRA$	=	incremental area of total cracking during the analysis year (% of total carriageway area)
ΔRDS	=	incremental increase in the standard deviation of rut depth during the analysis year (mm)
FM	=	freedom to manoeuvre index based on carriageway width in m and AADT
NPT_a	=	number of potholes per km at the start of the analysis year
ΔNPT	=	incremental number of potholes per km during the analysis year
TLF	=	time lapse factor
AGE3	=	pavement age since last overlay (rehabilitation), reconstruction or new construction (years)
m	=	environmental coefficient
K_{gm}	=	calibration factor for environmental coefficient; and RI_a is the roughness at the start of the analysis year (IRI in m/km)
RI_a	=	roughness at the start of the analysis year (IRI in m/km)

In the current research, HDM models were not used as the main aim of the study is to develop new deterioration models for flood affected pavements using data from Australian pavements.

2.12 NETWORK LEVEL STRUCTURAL DETERIORATION MODELS

Austroads (2010b) developed the network level interim structural deterioration models based wholly on the measured FWD deflections at the monitored sections to represent deterioration. The deterioration models were developed to estimate the pavement/subgrade strength variable in predicting functional (roughness and rutting) deterioration and to predict the change in pavement/subgrade strength with increasing pavement age. Functional distress variables, such as roughness, rutting and cracking, were not considered in the structural deterioration model development even though these distresses could be seen as being integral with structural deterioration and manifestations of structural deterioration. At a project level, it may be possible to relate specific local factors such as seasonal and drainage effects and cracking and potholing to changes in structural strength, however, at a network level these details are neither relevant nor capable of being modelled in any meaningful way.

The strength deterioration analysis was undertaken in terms of the Modified Structural Number, SNC as it was considered to represent the overall strength of the pavement/subgrade system, rather than the measured deflection. SNC historically has been used to represent the strength of the pavement and subgrade system in predicting pavement performance at a network level. The development of network level structural deterioration models for asphalt and sealed unbound granular pavements used normalised strength data, the ratio of current to initial strength, i.e., $\mathrm{SNC_i/SNC_o}$, termed the Strength Ratio, $\mathrm{SNC_{ratio}}$, which became the dependent variable for structural model development (Austroads 2010b).

The structural deterioration models for asphalt and unbound granular pavements are reasonably reliable for pavement sites where a decreasing trend of Thornthwaite Moisture Index (TMI) (Thornthwaite 1948) occurs caused by reducing annual rainfall. Most pavement performance was observed during long term drying conditions which caused most pavement/subgrades to gain strength which was observed by decreasing deflections since 2000. There is some evidence of a loss of pavement/subgrade strength in the wheel paths relative to the pavement/subgrade strength between the wheel paths from the deflection data (Austroads 2010b). Hence, in the development of network level structural deterioration models, regardless of whether pavement age and/or the TMI are used as independent variables, the analysis should be performed on a large range of data over a long period of time instead of focussing on individual sections over a relatively short monitoring period (Austroads 2010b).

The network level structural deterioration models developed for asphalt and unbound granular pavements, with the SNC_{ratio} as the dependent variable, are as follows:

$$SNC_{ratio} = 0.991 \times (2 - EXP(0.00132 \times TI_i + 0.256 \times \frac{AGE_i}{DL}) \quad (2.22)$$

$$SNC_{ratio} = 0.9035 \times (2 - EXP(0.0023 \times TI_i + 0.1849 \times \frac{AGE_i}{DL}) \quad (2.23)$$

Equation (2.22) is for asphalt pavement and Equation (2.23) is for sealed unbound granular pavements, where,

SNC_{ratio}	=	current strength of pavement/subgrade relative to its initial strength $(=SNC_i/SNC_0)$
SNC_i	=	modified structural number at the time 'i' of measurement
SNC_0	=	modified structural number at the time of the pavement construction (AGE = 0)
TI_i	=	Thornthwaite Moisture Index at the time 'i' of assessment
AGE_i	=	age of pavement (number of years since construction or last rehabilitation
DL	=	pavement design life (years).

The above interim structural deterioration models predict that the impact of the TI variable is greater on the deterioration of unbound granular pavements than for asphalt pavements (Austroads 2010b).

2.13 FUNCTIONAL ROAD DETERIORATION MODELS

Mechanistic-empirical deterministic-based pavement deterioration models for rutting and roughness and the structural deterioration of typical good quality sealed granular pavements were developed that are not liable to rapid deterioration at a network level in Australia. All of these models were based on observational data collected from LTPP/LTPPM (Long-Term Pavement Performance/Long-Term Pavement Performance

Maintenance) sites on arterial road sites across Australia from 1994 to 2007. The observational data were extended by experimental deterioration data from a series of full-scale simulation experiments using accelerated load testing (ALT) on test pavements (Martin 2009). The interim level functional road deterioration models considered Traffic load, MESA (millions of equivalent standard axles/lane/year), Pavement age, AGE (years), Thornthwaite Moisture Index (TMI), Initial pavement/subgrade strength (SNC_o), Annual average maintenance expenditure, me ($/lane-km), Cumulative rutting, roughness and cracking as independent variables (Austroads 2010a). Interim network level functional road deterioration models were developed for rutting, cracking and roughness of sealed granular pavements. These road deterioration models are applicable for the gradual deterioration phase of sealed granular pavements which represents the usual range of conditions for in-service pavements. A definition of the limit to the gradual deterioration phase was determined based on Accelerated Load Testing (ALT) experimental deterioration data from testing various surface maintenance treatments on sealed granular pavements (Austroads 2010a).

These mechanistic-empirical deterministic deterioration models for roughness, rutting and structural deterioration are applicable at a network level for the strategic life-cycle costing analyses of typical sealed granular pavements. The roughness deterioration models can also be used in the estimation of heavy vehicle road wear charges and other network level applications (Martin 2009). During the normal life cycle performance of typical deteriorating flexible pavements there are three phases of deterioration designated as

- Initial densification
- Gradual permanent linear deterioration
- Rapid non-linear deterioration leading ultimately to catastrophic failure.

The above phases were simulated by accelerated load testing (ALT) testing of the various surface maintenance treatments and formed the basis for defining the frontier limiting the gradual deterioration phase of sealed granular pavements. This frontier was defined as follows in Equation (2.24), based on a binary logistic regression analysis of the samples identified undergoing gradual deterioration and those undergoing rapid deterioration (Austroads 2010a; Martin 2009). Using the total of 140 cumulative roughness deterioration dependent variable samples, ΔIRI, and their associated independent variables from the LTPP/LTPPM (Long-Term Pavement Performance / Long-Term Pavement Performance Maintenance) sites, a cumulative roughness deterioration model, based on the linear addition of individual contributing cumulative distress components, was developed from this data for the gradual deterioration phase as defined by the following Equation:

$$rut_{max} = 86.347 - 11.008 \times IRI \quad (2.24)$$

Where,

rut_{max} = mean maximum vertical deformation from the original surface profile (mm) with an absolute maximum value of 25 mm

IRI = mean maximum vertical deformation from the original surface profile (mm) with an absolute maximum value of 25 mm

Predicting pavement deterioration beyond the gradual deterioration phase is not reliable due to a lack of observational data. In addition, the distress during the rapid deterioration phase would not be acceptable to the road users; consequently all the deterioration models developed are applicable only for the gradual deterioration phase (Martin 2009).

$$\Delta rut = k \times (AGE_i - 1)^{0.617} \times \{0.022 \times (100 + TI_i)/SNC_0 + 0.594 \times MESA - 0.000102 \times me\} \quad (2.25)$$

where,

Δrut = cumulative rut depth (mm) after initial densification at $AGE_i = 1$

AGE_i = number of years ‘i’ since construction or last rehabilitation

TI_i = Thornthwaite Moisture Index for climate pavement conditions at year ‘i’

SNC_0 = modified structural number for pavement/subgrade strength (years) at $AGE_i = 0$

MESA = annual traffic load per lane in millions of equivalent standard axles per lane

me = annualised pavement maintenance expenditure ($/lane-km/year)

k = calibration coefficient for local conditions (default value = 1.0)

The total rut depth, rut (mm), is estimated as follows:

$$rut = R_0 + \Delta rut \quad (2.26)$$

The model for cumulative roughness deterioration, ΔIRI, in terms of IRI (m/km), was defined as follows (Austroads 2010a):

$$\Delta IRI = kr \times [196.74 \times STRUC + 0.016 \times \Delta crx + 0.25 \times \Delta rut + 0.972 \times ENVIR] \quad (2.27)$$

where,

ΔIRI	=	cumulative increase in roughness, IRI (m/km), from the initial roughness, IRI_0, at zero pavement age, AGE_0
STRUC	=	$EXP[m \times AGE_i] \times MESA \times AGE_i \times [1 + (SNC_o - 0.0000758 \times \Delta crx \times B \times S]^{-5}$
		$= EXP[m \times AGE_i] \times MESA \times AGE_i \times [1 + SNC_o] – 5$ (No cracking)
S	=	nominal maximum size (mm) of seal aggregate
B	=	factor for estimating the field layer thickness (FLT) of bitumen binder
		= 0.6 for single seals
		= 0.9 for double seals
Δcrx	=	cumulative percentage (%) area of surface cracking (0 to 100%) contribution to roughness deterioration
Δrut	=	cumulative rut depth (mm) after initial densification at $AGE_i = 1$
EXP	=	e raised to the power
ENVIR	=	$m \times IRI_i \times AGE_i$
m	=	environmental coefficient $= 0.0197 + 0.000155 \times TI_i$
TI_i	=	Thornthwaite Moisture Index for climate pavement conditions at year ‘i’
AGE_i	=	number of years ‘i’ since construction or last rehabilitation
SNC_0	=	pavement/subgrade strength (years) at $AGE_i = 0$
MESA	=	annual traffic load per lane in millions of equivalent standard axles
IRI_0	=	initial roughness, IRI (m/km), at zero pavement age (typical range 1.0 to 1.8)
kr	=	calibration coefficient for roughness (default = 1.0).

The term, STRUC, was derived from the HDM-4 incremental roughness model (Morosiuk et al. 2001) as it was found to be the most appropriate form for the traffic load component for cumulative roughness.

2.14 DETERIORATION MODELS FOR SEALED LOW VOLUME ROADS

ARRB (Australian Road Research Board) initiated a study in 2000 to develop both sealed and unsealed mechanistic-empirical (ME) deterministic road deterioration (RD) models suitable for Australian local road conditions, using all data collected during the monitoring period from 2002 to 2009 from all states and territories. Development of the structural deterioration model was based on a non-linear form involving the pavement age, expected design life (DL), climate and traffic independent variables. It was found that Equation (2.28) for the strength ratio, $\mathrm{SNC_{ratio}}$, the dependent variable defining structural deterioration, was the most appropriate for the selected data set (Martin et al. 2011).

$$\mathrm{SNC_{ratio}} = \mathrm{ks} \times 0.919 \times \left[2 - \mathrm{EXP}\left(0.242 \times \frac{\mathrm{AGE_i}}{\mathrm{DL}} + 0.507 \times \mathrm{MESA}\right)\right] \quad (2.28)$$

where,

$\mathrm{SNC_{ratio}}$ = $\mathrm{SNC_i / SNC_o}$

$\mathrm{SNC_i}$ = modified structural number of the pavement/subgrade strength at time of pavement age, $\mathrm{AGE_i}$
$= 3.2 \times \mathrm{D_{oi}}^{-0.63}$ (Paterson 1987)

$\mathrm{D_{oi}}$ = maximum deflection measured at pavement age, $\mathrm{AGE_i}$, using a Falling Weight Deflectometer (FWD) or Heavy Weight Deflectometer (HWD)

$\mathrm{SNC_o}$ = initial modified structural number of the pavement/subgrade strength at pavement age, $\mathrm{AGE_i} = 0$

ks = local calibration factor for structural effects (default = 1.0)

EXP = exponential function

$\mathrm{AGE_i}$ = pavement age, the lesser of the number of years ‘i’ since construction or last rehabilitation

DL = expected (assumed) design life of the pavement (years).

2.14.1 Model for Rutting Deterioration

The observed rut depth (mm) was the average of the maximum rut depths measured in the inner and outer wheel path of the lane for a given lane length. The rut depth was measured using a multilaser profiler (MLP) with a 2 m simulated straight-edge across each wheel path (Martin et al. 2011).

Similar to the treatment of structural deterioration for all unbound granular base pavements, the data sets, whether surfaced with asphalt or sprayed seal, were pooled in a single data set and analysed together to develop the rutting deterioration model (Martin et al. 2011). The pavement rut depth, R_i, as described by Equation (2.29), consists of two components: (i) the initial rutting, R_o, which normally occurs during the initial densification period of one year after the pavement construction or rehabilitation; and, (ii) the cumulative rutting deterioration, Δrut, which develops after initial rutting to be the remaining part of the observed total rut depth (Martin et al. 2011).

$$R_i = R_o + \Delta rut_i \tag{2.29}$$

where,

R_i	=	observed rut depth (mm) at time i
R_o	=	initial densification rutting (mm) value at the end of year 1
Δrut_i	=	cumulative change in rut depth since initial densification (mm) at time i
		$= K_{rid} \times [51740 \times (MESA \times 10^6)^{(0.09\ +\ 0.0384 \cdot 6.5 \cdot SNC_0\text{^}{-1.6})} \times SNC_0^{-0.502} \times COMP^{-2.3}]$
in which, K_{rid}	=	calibration factor for initial densification (= 1.0 default for single seals and asphalt)
MESA	=	million ESA/lane/year
COMP	=	relative compaction value (100% assumed)
SNC_0	=	initial modified structural number

A number of independent variables were considered for predicting cumulative rutting deterioration, Δrut_i. These included the pavement age, climate (TMI), traffic loading and SNC_i. A variable for the impact of maintenance on rutting was also included by estimating the annualised expenditure on surfacing works. These independent variables were all used in a range of pre-formulated models involving linear and non-linear functions. These functions were trialled and the analysis outputs reviewed. Consequently, it was found that the following cumulative rutting deterioration model, Equation (2.30), was the most suitable model for the observational data despite a poor goodness of fit ($r^2 = 0.07$) to the highly variable cumulative rutting data.

$$\Delta rut_i = \text{kr} \times 4.003 \times [0.0035 \times \text{AGEi} + 0.18 \times (100 + \text{TMI})/100 + \text{EXP}\ (5.853 \times \text{MESA} - 0.418 \times \text{SNCi})] \quad (2.30)$$

where,

k_r = local calibration factor for rutting (default = 1.0)

TMI = Thornthwaite Moisture Index (incorporates temperature and rainfall impacts) and all other variables are as defined previously.

2.14.2 Model for Cracking Deterioration

Although this study could not develop any model for cracking due to the unavailability of relevant data for flood affected roads, cracking model from Martin et al. (2011) is included as it was needed to describe the roughness deterioration model of Martin et al. (2011).

The most suitable cumulative cracking deterioration model for the full range of cracking up to 100% was described by Martin et al. (2011) as in Equation (2.31).

$$\Delta crx = 100 - 200 \times [1 + \text{EXP}\ (k_c \times (\text{Sage}/((200 - \text{TMI})/25))^{0.649})]^{-1} \quad (2.31)$$

where,

Δcrx = cumulative total cracking as a percentage of observed lane area (%)

k_c = local calibration factor for cracking (default = 1.0)

Sage = elapsed time after crack initiation (years)

= age of seal – seal life (estimate)

seal life = $[(0.158 \times T_{min} - 0.107 \times R + 0.84) / (0.0498 \times T_{ave} - 0.0216 \times D - 0.000381 \times S^2)]^2$

T_{ave} = the average temperature of the site (°C)

= $(T_{max} + T_{min})/2$

T_{max} = the yearly mean of the daily minimum air temperature (°C)

T_{min} = the yearly mean of the daily maximum air temperature (^{o}C)

R = risk factor associated with delaying resealing (range of 1 to 10; used R = 10 for high

2.14.3 Model for Roughness Deterioration

The observed roughness was measured in terms of the International Roughness Index (IRI, m/km) using the MLP. Roughness for a given lane length was the average of the inner and outer wheel path roughness values (Martin et al. 2011). The total roughness, IRI_i, at any time, is defined as follows by Equation (2.32):

$$IRI_i = IRI_o + \Delta IRI_i \tag{2.32}$$

where,

IRI_o = initial roughness (m/km) at pavement age, $AGE_i = 0$

ΔIRI_i = cumulative roughness at pavement age, $AGE_i = i$

and all other variables are as defined previously.

Equation (2.33) helps to predict cumulative roughness deterioration with a full set of contributing component variables for traffic, rutting, cracking and climatic effects (Martin et al. 2011):

$$\Delta IRI_i = k_{iri}\,[a_1 \times IRI_{env} + a_2 \times \Delta rut_i + a_3 \cdot \Delta crx + a_4 \times IRI_{struc}] \tag{2.33}$$

The final cumulative roughness deterioration model is described by Equation (2.34):

$$\Delta IRI_i = k_{iri}\,[1.393 \times IRI_{env} + 0.09 \times \Delta rut_i + 0.029 \times \Delta crx] \tag{2.34}$$

where,

IRI_i = cumulative increase in overall roughness since the initial roughness, IRI_o

ΔIRI_i = cumulative roughness at pavement age, $AGE_i = i$

IRI_{struc} = cumulative roughness due to traffic (m/km)

$= 134 \times EXP(m \times AGE_i) \times MESA \times AGE_i \times (1 + SNC_o - 0.0000758 \times B \times S \times \Delta crx)^{-5}$

m = $0.0197 + 0.000155 \times TMI$

k_{iri} = local calibration factor for roughness (default = 1.0)

B	=	0.6 for single seal and 0.9 for double seal surfacing
		=30 mm for asphalt (assumed)
S	=	0.6 for single seal and 0.9 for double seal surfacing
		=30 mm for asphalt (assumed)
IRI_{env}	=	cumulative roughness due to climatic effects (m/km)
		$= m \times IRI_o \times AGE_i$
a_1, a_2, a_3 & a_4	=	regression model coefficients
Δrut_i	=	cumulative increase in rutting, (Equation (2.29))
Δcrx	=	cumulative increase in cracking, (Equation (2.30)).

And all other variables are as previously defined.

2.15 IMPACT OF MOISTURE ON PAVEMENT

The environmental factors which significantly affect pavement performance are moisture and temperature. Moisture ingress into a pavement is the single most destructive environmental influence (BCC 1993; BCC 2011). Impact of moisture on pavement was reviewed initially in this research. Although due to lack of field data on moisture, this research could not include moisture information in the analysis.

To avoid moisture-related problems, a major objective in pavement design should seek to prevent the subbase, subgrade, and other susceptible paving materials from becoming saturated, or even exposed to constantly high-moisture levels. The three common approaches for controlling or reducing the problems caused by moisture include (Schaefer et al. 2008):

- Preventing moisture from entering the pavement system.
- Using materials and design features that are insensitive to the effects of moisture.
- Quickly removing the moisture that enters the pavement system.

No single approach can completely negate the effects of moisture on the pavement system under heavy traffic loading over many years. For example, it is practically impossible to completely seal the pavement, especially from moisture that may enter from the sides or beneath the pavement section. While materials can be incorporated into the design which are insensitive to moisture, this approach is often costly and in many cases not feasible (e.g. may require replacing the subgrade). Drainage systems also add costs to the road, as maintenance is required to maintain drainage systems as well as to seal systems for effective performance over the life of the system. Thus, it is often necessary to employ all approaches in combination for critical design situations (Schaefer et al. 2008).

As moisture builds up in and between asphalt layers as well as in unbound layers, positive pore pressures develop as wheel loads are transmitted to the moisture-affected layers. Since water is incompressible, pressure is subsequently transferred to the surrounding material. This can cause lifting, layer separation (delamination), fatigue cracking and subsequent potholing. Poorly-drained asphalt surfaces are prone to aggregate stripping and ravelling (BCC 2011).

The strength (e.g. CBR) of a granular pavement material is related to: (a) the use of hard aggregate stone which will not break down under load, and (b) strong inter-particle (mechanical) interlock derived from the angular shape and low plasticity. This will also guarantee a high compacted density and low moisture content (BCC 2011).

When a granular pavement layer becomes saturated its strength can be severely compromised, to the extent that mechanical interlock is reduced (particularly when the material has moderate plasticity, e.g. PI (Plasticity Index) > 9). In addition, saturation introduces an apparent cohesion between particles by capillary attraction which can also produce high pore pressure (or low effective stress) and, consequently, low shear strength (BCC 2011).

The presence of moisture in a subgrade affects both the design thickness of a new or rehabilitated pavement and also long-term pavement performance. In the first instance a poorly-drained or low-CBR subgrade will require a much greater thickness of granular layer cover than a pavement with a drier and higher-CBR subgrade. In addition, where expansive subgrades are encountered, variations in moisture content will also result in a change in volume (shrink/swell) and a change in subgrade strength. Problems associated with the types of expansive clays commonly encountered in Brisbane, particularly in the northern coastal suburbs, are usually related to surface cracking and loss of shape caused by volume changes in the subgrade (BCC 2011).

An alteration of the subgrade moisture content can result in a change in volume and/or a change in strength. The sensitivity of the subgrade strength/modulus to changes in moisture content should in all cases be assessed. The significance of these changes will depend on their magnitude and the nature of the subgrade material (Austroads 2012). Whilst the sensitivity of a subgrade to moisture, and its effect on strength and volume changes, are variable and related to soil type, in general the following will apply (BCC 2011) (Austroads 2012):

- for sandy soils, small fluctuations in moisture content produce little change in volume or strength
- for silty soils, small fluctuations in moisture content produce little change in volume but may produce large changes in strength
- for clay, at a low moisture content, small fluctuations in moisture may produce large variations in volume; if the moisture content is above the plastic limit, then large changes in strength may also occur.

2.15.1 Evaluation of Moisture Content

The main factors determining moisture conditions in soil are rainfall and evapotranspiration. It is suggested that a climatic classification, based on the moisture balance, may be useful in connection with estimating the probable moisture conditions in road subgrades and in the assessment of the performance of road-making materials in areas with differing climates. Where a ground water-table is present near the surface, however, this becomes the dominating factor controlling the moisture distribution under sealed surfaces (Russam and Coleman 1961).

The drainage of the pavement and subgrade layers has an over-riding influence on pavement performance. Many pavement designs and materials will perform quite adequately provided moisture is controlled. However, if moisture is allowed into the pavement, rapid deterioration can occur. In undertaking a pavement evaluation it is important to determine the adequacy of the current surface and sub-surface drainage system in preventing moisture infiltration into the pavement or, if water does penetrate, how it may be removed (BCC 2011).

TMR (2012) recommended that the influence and potential influence of surface and the sub-surface water must be considered during the assessment of an existing pavement and designing rehabilitation treatments. Often a large portion of expenditure on pavement rehabilitation is related, at least in part, to moisture-related pavement distress. Rainfall records, both in terms of intensity, duration and distribution, may be of value when related to the original design assumptions and the development of pavement distress. High intensity falls may exceed the capacity of the drainage systems (e.g. stormwater, cross drainage culverts) leading to a “back up” of water into and through a subsoil drainage system. On the other hand, a prolonged wet period, even if intensities are low, may lead to a fall in subgrade strength through a general rise in moisture levels.

It is often possible to identify moisture-related distress in the existing pavement during a visual survey. Additional field or laboratory testing can also reveal where moisture is relatively high and how this affects pavement performance. The cause of any identified moisture-related distress needs to be addressed if any rehabilitation treatment is to achieve its intended purpose. Examples of distress caused by the ingress of water are as follows (TMR 2012):

- in pavements with asphalt layers, stripping, rutting, loss of surface shape (depressions), fatigue cracking and potholes; and
- in rigid (concrete) pavements, pumping, the formation of voids, cracking, joint deterioration and corner breaks.

In unbound pavement materials, prolonged exposure to excess moisture results in moisture-accelerated distresses. These distresses are primarily initiated by factors other than moisture (e.g. traffic loading) but whose rate of deterioration is accelerated when accompanied by the presence of moisture. Prolonged exposure to excess moisture may lead to low subgrade bearing capacity, reduction in stiffness of unbound granular layers, degradation of material quality and loss of bonding between the pavement layers (Salour and Erlingsson 2014).

Subgrade soil strength and stiffness are major factors in the design and performance of the pavements, particularly low volume pavements. Variations in subgrade moisture, with corresponding changes in volumetric and mechanical properties of subgrade soils, may cause or accelerate pavement distress. The prediction of the subgrade moisture content is a complicated process because of the variability in soil properties and soil behaviour, under repeated traffic loads, environmental factors, geometric factors and site conditions and because of the complexity of moisture movement in soil. It is vital that the modulus chosen for the design accurately reflects the in situ moisture condition of the project site (Hall and Rao 1999).

The placing of a sealed pavement surfacing isolates the subgrade from some of the principal influences which affect moisture changes, especially infiltration of large quantities of surface water and evaporation. Where these influences are the controlling ones (i.e. drier environments), the moisture conditions in subgrades generally tend to remain relatively uniform after an initial adjustment period. In high rainfall areas, subgrade infiltration- particularly lateral infiltration through unsealed shoulders, through defects in wearing surfaces, or through joints- has a major influence on the subgrade moisture conditions. Specific actions should therefore be taken to guard against this influence (Austroads 2004).

A major design consideration in flexible pavement infrastructure design is to keep the base and subbase granular layers and subgrade material from being exposed to prolonged high moisture levels (Salour and Erlingsson 2014). The proximity of the ground water table or local perched water table to the pavement wearing surface may also play a significant role in influencing the subgrade moisture conditions. In circumstances where the height of the water table fluctuates seasonally, the subgrade moisture condition will reflect these fluctuations equally across the central and peripheral regions of the pavement (Austroads 2004).

In practice, approaches employed to reduce the undesirable effect of excess moisture presence in pavement structures are to incorporate design features to minimize moisture entering the system, quickly removing the moisture that has entered the system, and using less moisture-insensitive materials. An adequate pavement drainage system can effectively reduce any potentially detrimental effects of water. Surface and subsurface drainage is an important factor for long-term pavement performance of road networks exposed to environments with a high groundwater level, high precipitation, frequent freeze-thaw cycles and a poor subgrade condition. Application of subsurface drainage has gained popularity among road authorities over the past decades, which routinely require modern drainable systems to reduce moisture related issues (Salour and Erlingsson 2014).

Salour and Erlingsson (2014) conducted a study in an instrumented flexible pavement structure. The effect of groundwater level on the structural response of the pavement was investigated by conducting frequent FWD tests with multilevel loads. The test section was about 300 m long and located along county road 126 near Torpsbruk in southern Sweden. This section of the road was equipped with subsurface drainage system (vertical deep-drain sheets) on both sides of the road that could be manipulated to change the moisture condition of the pavement system. The drainage outlets were manually clogged during a three-month period in summer 2011, allowing the water table to rise and the unbound layers to undergo high moisture conditions. Thereafter the drainage outlets were unclogged, allowing the structure to regain its previous draining hydrological condition (Salour and Erlingsson 2014).

The pavement response was evaluated during the “draining”, “non-draining” and “transition” conditions. The “draining” condition was the period in which the drainage system was fully functioning. The “non-draining” condition was when the outlets were clogged and therefore the drainage was not functioning. The “transition” condition was the changeover period between these two conditions. The mechanical response of the pavement structure under these conditions was evaluated by conducting frequent Falling Weight Deflectometer (FWD) testing with multilevel loads (Salour and Erlingsson 2014).

The groundwater level increased sharply after the drainage clogging. Within a few days after clogging the drainage pipes, the sensors registered a 1.0 m increase in the groundwater table level. Thereafter a more gradual increase in the groundwater level was observed during the clogged drainage period. The response of the pavement moisture level to drainage condition was quite rapid. The moisture content in the lower subgrade levels (1.50 and 1.20 m depths) increased sharply after the drainage was clogged. The moisture content at the lower subgrade increased from the normal summer value of about 15% to nearly 40%. The moisture contents then remained almost steady during the whole period of the “non-draining” condition. It is very probable that the subgrade below 1.20 m reached a saturated state during the “non-draining” period (Salour and Erlingsson 2014).

Pavement stiffness decreased significantly when the base and the subgrade began to thaw. Changes in back-calculated layer moduli showed clear correlation with layer moisture content measurements. The field data showed a considerable decrease in the bearing capacity of the pavement structure when the highest annual moisture in subgrade was also registered. Both deflection basin indices and back-calculated layer moduli indicated that the pavement was weakest when the subgrade completely thawed. Thereafter, the pavement gradually regained its stiffness as the excess water drained out from the layers. Complete recovery of the pavement took more than one month. Back-calculations of the FWD data showed 63 percent loss in the subgrade modulus and 48 percent loss in granular base and subbase moduli, respectively, during spring thaw compared to the summer values (Salour and Erlingsson 2012).

2.15.2 Role of Drainage

Appropriately designed and constructed sub-surface drainage will enhance the performance of the pavement by controlling the effects of moisture. For example, a marginal base course may, under a dry operating environment, achieve a soaked CBR of 80%. However, if the material is saturated for any length of time (e.g. flooding) the CBR could be reduced by up to 50%. Therefore an appropriately designed, constructed and maintained drainage system is required to protect the pavement from potential distress such as loss of bearing capacity (shear failure or shoving) and deformation resistance (BCC 2011).

As sub-surface moisture has a high impact on pavement performance, sub-surface drainage should be considered when determining rehabilitation strategies. The effective maintenance of pavement drainage systems is as important as their design and construction. Maintenance tasks include the clearing of debris and growth from the channel and inlet and outlet pits, particularly after heavy rainfall, forest or bush fires, or in seasons when trees shed their leaves. Debris screens may be required in problem areas. Accumulated silt or drift sand in the culvert barrel must also be removed periodically by mechanical or hydraulic means (BCC 2011).

TMR (2012) recommended that the influence and potential influence of surface and the sub-surface water must be considered during the assessment of an existing pavement and designing rehabilitation treatments. Often a large portion of expenditure on pavement rehabilitation is related, at least in part, to moisture-related pavement distress. Rainfall records, both in terms of intensity, duration and distribution, may be of value when related to the original design assumptions and the development of pavement distress. High intensity falls may exceed the capacity of the drainage systems (e.g. stormwater, cross drainage culverts) leading to a "back up" of water into and through a subsoil drainage system. On the other hand, a prolonged wet period, even if intensities are low, may lead to a fall in subgrade strength through a general rise in moisture levels.

- It is often possible to identify moisture-related distress in the existing pavement during a visual survey. Additional field or laboratory testing can also reveal where moisture is relatively high and how this affects pavement performance.

The cause of any identified moisture-related distress needs to be addressed if any rehabilitation treatment is to achieve its intended purpose. Examples of distress caused by the ingress of water are as follows by TMR (2012):

- in rigid (concrete) pavements, pumping, the formation of voids, cracking, joint deterioration and corner breaks; and
- in pavements with asphalt layers, stripping, rutting, loss of surface shape (depressions), fatigue cracking and potholes.
- In some cases, a drainage assessment will be required as part of the overall project. Normally the designer undertaking the pavement rehabilitation investigation is concerned with information affecting pavement drainage. Except for pavement drainage, assessment of the adequacy of the drainage system is not usually within the scope of the pavement rehabilitation designer's investigation. If assessment of the drainage not directly related to the pavement is required, it is recommended that a specialist drainage designer, or designers, be engaged to undertake this part of the investigation.

Appropriately designed and constructed sub-surface drainage will enhance the performance of the pavement by controlling the effects of moisture. For example, under a dry operating environment, achieve a soaked CBR of 80%. However, if the material is saturated for any length of time (e.g. flooding) the CBR could be reduced by up to 50%. Therefore, an appropriately designed, constructed and maintained drainage system is required to protect the pavement from potential distress such as loss of bearing capacity (shear failure or shoving) and deformation resistance. As sub-surface moisture has a high impact on pavement performance, sub-surface drainage should be considered when determining rehabilitation strategies (BCC 2011).

Within the Brisbane City Council area, most subgrades encountered, and, to a large extent, granular pavement layers constructed are adversely affected by water. Strategies to negate or control the effect of water include the following (BCC 2011):

- Protection of the pavement from the ingress of moisture, i.e. with the provision of seal coats or impermeable barriers. If the presence of water is related to surface infiltration, then bituminous seals provide the best waterproofing characteristics of all the possible sealing options. Asphalt, because of its void content and a tendency to segregate, can be permeable; however, the permeability of asphalt is highly dependent on the type, grading, air void content and level of compaction.

- Rendering the pavement materials less sensitive to moisture ingress through stabilisation with lime and/or cement.
- The provision of sub-surface drainage systems.

Where the cost of providing a continuous drainage system is high, alternate considerations to limit the effect of moisture may be considered such as subgrade and pavement stabilisation. Whilst this is not a substitute for a good drainage system, it may be a practical outcome in some situations because it is a cheaper option (BCC 2011).

2.15.3 Rainfall and dry periods

Rainfall records, both in terms of intensity, duration and distribution, may be of value when related to the original design assumptions and the development of pavement distress. High intensity falls may exceed the capacity of the drainage systems (e.g. stormwater, cross drainage culverts) leading to a “back up” of water into and through a subsoil drainage system. On the other hand, a prolonged wet period, even if intensities are low, may lead to a fall in subgrade strength through a general rise in moisture levels. Conversely, an extended dry period may result in unexpected subgrade shrinkage with consequent damage to the pavement (TMR 2012).

2.16 SUMMARY

Pavements subject to flooding and inundated for a certain period of time go through a rapid deterioration phase rather than gradual deterioration as predicted by most of the available deterioration model. The models discussed in this chapter, were developed considering pavement will experience normal climatic conditions such as average rainfall and design traffic and hence, deterioration will be gradual. Therefore, none of the deterioration models considered gradual rise of flood water or rapid deterioration of pavements after flooding. Therefore, it will be hard to accurately predict the deterioration of flood affected pavements using these models. Data for before and after flooding of the pavement road sections are required to measure the structural performance of partial or fully saturated pavements. Previous models should also be considered too while developing models for such pavements.

The objective of this research is to develop mechanistic-empirical deterministic models to predict structural and surface condition (rutting and roughness) deterioration of flood affected pavements. This method is adopted because the research has field testing data before and after flooding.

3. CASE STUDIES ON DETERIORATION OF FLOOD AFFECTED PAVEMENTS

3.1 STUDIES ON DETERIORATION OF FLOOD AFFECTED PAVEMENTS

The diversity of extreme weather events impacting transportation is immense and each event poses unique challenges. They can create havoc with transportation asset management plans, which are often based on predictable deterioration curves (Condric and Stephenson 2011). Over the past several years, extreme weather has disrupted transportation systems in nearly every region of the United States. Derechos, snow storms, and intense hurricanes have plagued the east coast, while the Midwest has suffered massive and prolonged flooding. In the southwest, dust storms and wildfires have forced extended road closures and endangered drivers. Hurricanes Katrina, Irene and Sandy probably did more damage in a few days in the USA than normal condition deterioration on the nation's road network over decades (Condric and Stephenson 2011). Transportation agencies have decades of experience managing weather variability and are able to quickly and efficiently handle common weather disruptions. However, many road agencies are now having to manage disruptions from more frequent and intense events (Condric and Stephenson 2011).

This chapter reviews the literature assessing deterioration of flood affected pavements to understand the impact of flooding on pavements and the need for deterioration models for flood affected pavements. Very limited literature was available on the assessment of the impacts of flood on pavement deterioration. Studies that presented some findings on the damage assessment of flood affected pavements include Gaspard et al. (2007), Helali et al. (2008), Zhang et al. (2008), Condric and Stephenson (2011) and Chen and Zhang (2014). Studies such as Vennapusa et al. (2013) discussed geo-infrastructure damage assessment, repair, and mitigation strategies due to flood while Mallick et al. (2014) discussed the development of a methodology and tool for assessing the vulnerability of roadways to flood induced damage. Khan et al. (2015) discussed development of a post-flood road maintenance strategy using a case study of Queensland, Australia. Lee et al. (2014) discussed the use of stabilisation techniques to restore flood-affected roads in

Queensland. Studies presenting deterioration modelling considering extreme weather events, such as flooding or snow storms, are Shamsabadi et al. (2014), Khan et al. (2014a), Khan et al. (2014b) and Tari et al. (2015).

3.2 STUDIES ON THE IMPACT OF HURRICANES KATRINA AND RITA IN THE USA

In 2005, two devastating hurricanes, Katrina and Rita, hit the city of New Orleans and the south-eastern and south-western regions of Louisiana in the USA. Approximately 3,220 km of roadway in the Greater New Orleans area were submerged by floodwaters for up to five weeks. Immediately after the hurricanes, there was great concern in the state about the integrity of pavement structures in the flooded area due to the sustained flooding (Zhang et al. 2008). Although the pavements appeared unaffected, they may have suffered undetected damage in the roadbed soils that could result in failures at a later time when emergency federal funds were no longer available (Helali et al. 2008). Three separate studies published on the assessment of flood affected pavements following Hurricanes Katrina and Rita, Gaspard et al. (2007), Helali et al. (2008), Zhang et al. (2008). The study by Chen and Zhang (2014) assessed the surface condition deterioration of flood affected pavements.

The Louisiana Transportation Research Centre (LTRC) sent investigative teams to assess the flooding impact on pavement structures in the area. The LTRC initially conducted structural damage testing on several roads that were under construction to determine any damage that might require additional work. Based on the preliminary results, additional roads were tested in the New Orleans area. A total of eight roadways were tested using Falling Weight Deflectometer (FWD), Dynaflect, Dynamic Cone Penetrometer (DCP) and coring (Gaspard et al. 2007). The FWD deflection data were used to back-calculate the elastic moduli of the pavement layers. The other deflection measuring device, the Dynaflect, was used to determine the structural number (SN) and subgrade resilient modulus (M_r) of the tested pavements. The DCP provided verification of the base and subgrade readings; and the coring provided thicknesses and verification of moisture damage. Data from coring at different locations were used to verify the in situ pavement thickness and the integrity of pavement structure (Gaspard et al. 2007; Zhang et al. 2008).

3.2.1 *Assessment by Gaspard et al. (2007)*

Gaspard et al. (2007) evaluated data obtained under contract to Fugro to conduct structural testing of 383 km (238 miles) of state highway pavements in the greater New Orleans area at 0.16 km (0.1 mile) intervals. The results examined three structural parameters, D_1, SN_{eff} and M_r. The data was divided into those pavements that were submerged under water for periods over three days and pavements that were not submerged. This is not to imply that non-submerged pavements were not damaged by the hurricanes. The number and overweight loading of debris haul trucks immediately after the storms up until the time of testing several months later exacted a toll on the roadway system. Additional debris hauling also caused some damage.

Gaspard et al. (2007) could not examine visual distress and smoothness data, such as taken from PMS, as the post-hurricane data were not available when their report was conducted. Their findings are included in this section as follows.

- Overall, pavements that were submerged were found to be weaker than non-submerged pavements for each of the strength parameters tested. There was a difference in strength values for each of the pavement types, asphalt concrete, PCC and composite pavements evaluated.
- For the asphalt pavements, each of the strength parameters was weaker for the submerged pavements. Also there was a difference in these parameters depending on the thickness of the pavement.
- The variation in the thinner asphalt pavements was very high for the maximum deflection parameter indicating that any future rehabilitation or reconstruction design should be completed on a project by project basis.
- The duration of submergence was not a factor for the asphalt pavements. Damage was sustained regardless of the length of time the pavement was submerged.
- The overall equivalent strength loss for the asphalt pavements is similar to two inches of new asphalt concrete. Note that the thinner pavements required more asphalt concrete than the thicker pavements.
- PCC pavements demonstrated little relative loss of strength between those pavements that were submerged and the non-submerged pavements. While not significantly different, there was a reduced SN_{eff} for the submerged pavements.

- Similarly, duration of submergence was not a factor for the PCC pavements. As could be assumed, there was a difference in strength parameters based on thickness.
- The M_r for PCC pavements was similar between the submerged and non-submerged pavements. In general, the M_r for the PCC pavements was higher than the asphalt pavements.
- Although the loss of strength of the PCC pavements was minimal, other factors such as pavement smoothness might require a thicker overlay.
- The composite pavements demonstrated no need for additional structure in the pavement layers due to submergence. However, a weaker subgrade for the submerged areas is equivalent to 0.9 inches of asphalt concrete.

3.2.2 Comparison of Flooded and Non-flooded Pavements by Zhang et al. (2008)

The lack of structural data on pre-Katrina pavement structures made it impossible to conduct a comprehensive "before and after" style structural analysis to determine the reduction in the strength of pavement layers caused by flooding. As such, an alternative strategy, based on "flooded versus non-flooded," was used to determine the flooding impact on pavement structures by Zhang et al. (2008). This was an indirect and more complicated approach and relied heavily on a spatial style of analysis wherein areas that did flood, such as in Orleans and St. Bernard Parishes, were compared to areas that did not flood, such as Jefferson Parish. The critical issue in this approach was to establish a valid base on which a comparison could be made. Such validity was established in this analysis as follows. The Tri Parish area, Jefferson, Orleans, and St. Bernard, developed historically side by side and have similar geographic and geological conditions. Because of this, their highway traffic, design, construction, and maintenance history are similar. Having similar environmental conditions, pavement structures, and traffic volumes, it is reasonable to infer that differences in pavement strength between areas that flooded and areas that did not flood were due to submergence by floodwaters and flooding potential between adjacent areas (Zhang et al. 2008).

The analytical results of the field test data were grouped and analysed according to the pavement types: Asphalt Concrete (AC), Portland Cement Concrete (PCC), and composite. Table 3.1 indicates that for AC and PCC, the flooded pavements had a higher deflection, D_0 lower structural number, SN_{eff}, and lower subgrade resilient modulus, M_r, when compared with the values for non-flooded pavements. This indicated that for AC and PCC, the flooding did weaken the pavement structures. For the composite pavement, however, there were contradictory results in the SN_{eff} values [refer to the last two columns of Table 3.1 (Zhang et al. 2008)].

Table 3.1: Flooding Impact, Direct Comparison (Zhang et al. 2008)

Parameter	**AC**		**PCC**		**Composite**	
	Flooded	**Non-flooded**	**Flooded**	**Non-flooded**	**Flooded**	**Non-flooded**
D_0 (0.001in.)	10.12	6.98	5.01	4.51	6.18	5.31
SN_{eff}	6.18	7.28	7.92	8.39	8.3	8.21
M_r (ksi)	5.41	6.74	5.53	5.6	4.89	6.36
Note: 1 in. = 2.54 cm, 1.0 ksi = 6.9 MPa.						

Limited "before and after" data collected by the Louisiana Department of Transportation and Development (LA DOTD) indicated the detrimental impact of the flooding caused by Hurricane Katrina on submerged pavement structures in New Orleans. This method, however, could not be employed on other routes in the area due to a lack of data coverage. As such, a spatial analysis was employed where flooded areas were compared to the adjacent non-flooded areas based on the same engineering and environmental history and pavement conditions in these areas. The research indicated that field testing data collected with GPS (Global Positioning System) and processed with GIS (Geographical Information System) technology is a very effective way toward improved data management, processing, and analysis with respect to time and man power savings (Zhang et al. 2008).

The results of this study showed that, from the statistical network wide point of view, the flood waters did weaken AC pavement structures by reducing the stiffness of both the AC layer and subgrade in the submerged New Orleans area. AC pavements at lower elevations were affected more by flooding than ones at higher elevations. Meanwhile, the flooding impact on PCC structures was very limited in comparison to the AC results. No conclusion could be drawn with respect to flooding damage on composite pavement due to the variety of pavement structure, composition, and materials in that group (Zhang et al. 2008).

One particularly important observation from this study that was extremely relevant for city-parish roads, was that thinner pavements suffered proportionately more than their thicker, more robust counterparts as a result of exposure to flooding. The significance of this for the city parish lies in the consideration that they maintain a far greater number of lane-miles than does the LA DOTD within the impacted region and in the fact that those lane-miles are characterized by thinner pavements. This means that the city will very likely experience a massive failure in its transportation network at the moment that commerce in the city begins to show real recovery (Zhang et al. 2008).

3.2.3 Damage Assessment by Helali et al. (2008)

Helali et al. (2008) assessed the extent of damage to Jefferson Parish's road pavements caused by hurricanes Katrina and Rita. The study involved selecting a study area with a total road length of 338 miles, which represents approximately 20% of the Jefferson Parish road network, using an approach that involved; estimating the pre-Katrina/Rita pavement conditions; evaluating the post-Katrina/Rita pavement conditions; and, analysing the pavement damage (Helali et al. 2008). The pavement damage analysis involved both functional and structural analyses performed at the network level, section level, and the roadway level. The network level analysis involved identifying control sections (non-flooded sections) and comparing them with flooded sections, holding all other variables constant. The comparisons were performed using statistical techniques such as t-test and Analysis of Variance (ANOVA) (Helali et al. 2008)

One important factor in the assessment of the pavement damage is the existence of historical data that would be used as a benchmark for the before-and-after analysis. Jefferson Parish PMS database includes historical data for the majority of the pavement sections within the Parish's jurisdiction. This data includes historical condition data, both functional and structural, in addition to other attribute data such as as-built, geometric data, etc. The historical functional condition data was available for almost the entire network, while the structural data was only available for some sections. Therefore, the sections that had historical deflection data were selected, when possible, within the scope of the project such that both pre-Katrina and post-Katrina structural data would be available for those sections (Helali et al. 2008).

The **section level analysis** involved evaluating the rate of deterioration (RD) of the flooded sections versus the non-flooded sections (control sections), by comparing the predicted performance of each section, based on the available historical data and the performance prediction models, to the actual measured performance in 2007. The performance prediction models considered in the study included models available in

Jefferson Parish's PMS and models used by the Louisiana Department of Transportation and Development for the state maintained roads (Helali et al. 2008).

The **roadway level analysis** showed that on the roadways that were partially flooded, the flooded sections had higher deflection values (weakened structural condition) than the sections that were not flooded, indicating possible damage due to flooding. As an example, Figure 3.1 shows the entire roadway was historically (pre-Katrina) structurally homogeneous, however; post-Katrina data shows that the flooded sections seem to have higher deflection values, indicating possible damage from the flooding (Helali et al. 2008).

The **roadway level analysis** focused on analysing special cases of partially flooded roads, where some sections of the road were flooded, while other sections on the same road were not flooded. The main advantage of this approach is that it can potentially show the impact of the flooding on the pavement condition as compared to an identical section of the road that was not flooded. (Helali et al. 2008). The analysis also showed that on some of the roadways that were partially flooded, the flooded sections had worse functional condition than the sections that were not flooded, indicating possible damage due to flooding (Helali et al. 2008).

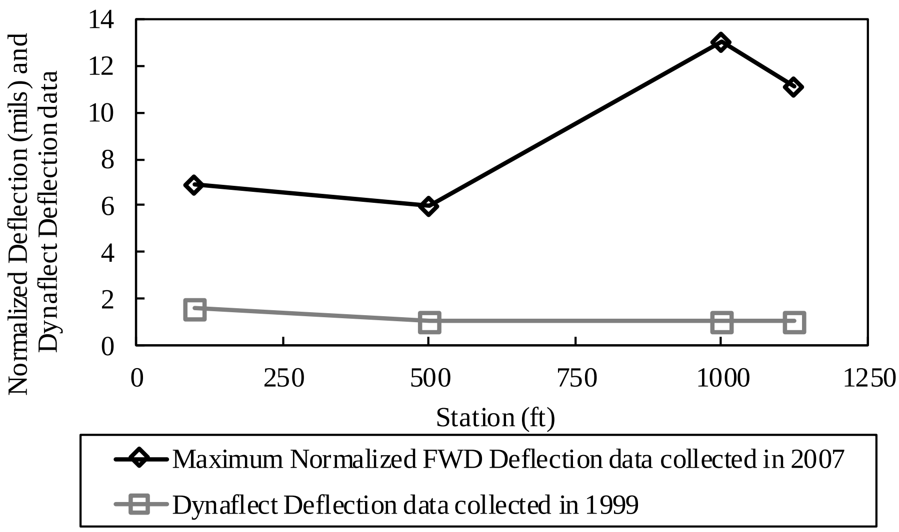


Figure 3.1: Comparison of pre- and post-Hurricane Katrina deflection data (Helali et al. 2008)

At the **network analysis level**, the general pavement condition in terms of D_0 (first sensor deflection), SN_{eff} (effective structural number), and M_r (subgrade resilient modulus) collected in 2007 was statistically analysed. For flexible pavement sections, the flooded pavement sections were in a significantly worse structural condition than the non-flooded roads, at a 95% confidence level. When analysing the flexible pavement sections as one group by ignoring the various differences in the other attributes, the control sections seemed to be significantly structurally stronger than the flooded sections. On average, the difference in SN_{eff} between control sections and flooded sections for all the flexible pavement sections was 1.0 SN unit, which is equivalent to 2.3 inches of AC material. Furthermore, when the sections with core data were analysed separately, the difference between the flooded sections and the non-flooded sections was on average 3.6 SN units, which is equivalent to 7.6 inches of AC (Helali et al. 2008).

Distress data was analysed by evaluating the rate of deterioration for key distress indexes during the period between the pre-Katrina data and the post-Katrina. Rate of deterioration of these indexes were statistically analysed to examine whether there was a significant difference between the flooded and non-flooded sections. This analysis showed that for AC pavements alligator cracking, transverse cracking, map cracking, longitudinal cracking and distortion had a significantly steeper rate of deterioration for the flooded routes than the non-flooded routes, which might indicate that the flooding accelerated the deterioration of the pavement. For PCC pavements, only transverse cracking and distortion/heave/depression distresses showed a significant difference in terms of the rate of deterioration. However, these distresses can be directly related to the change of subgrade support due to flooding (Helali et al. 2008).

3.2.4 Assessment of Damage to Surface Condition by Chen and Zhang (2014)

Chen and Zhang (2014) selected IRI and rutting to assess the pavement performance deterioration in District 02 before and after Hurricanes Katrina and Rita. For asphalt and composite pavements, the average IRI values and IRI increments in flooded zones were slightly higher than those in non-flooded zones. For concrete pavements, the average IRI values and IRI increments in flooded zones were slightly lower than those in non-flooded zones. Overall, the average IRI values and IRI increments in flooded zones were slightly higher than those in non-flooded zones. For asphalt pavements, the average maximum rutting depth for the 0.1 mile subsection and increments in flooded zones were slightly lower than those in non-flooded zones. In composite pavements, the average maximum rutting depth for the 0.1 mile subsection values and increments in flooded zones were slightly higher than those in non-flooded zones.

The PMS data illustrated increased damage to highways as a result of heavy trucking or vehicle loading required to transport the vast amounts of debris following the hurricanes. In addition, the data established an escalation in deterioration occurring as subgrade components were not initially designed to sustain such vehicle loads and may have been further weakened as roadways were submerged in water for extended periods of time (Chen and Zhang 2014).

3.3 STUDY ON THE FLOOD AFFECTED ROADS OF BRISBANE CITY COUNCIL

Brisbane City Council covers an area of approximately 1,367 km^2. It is home to almost 1,000,000 people. The Council road network ranges from minor residential roads to major arterial roads. Brisbane City Council, Australia's largest local government authority, has responsibility for a road network 5,657 km in length and the pavements have a replacement value of AU$3.63 billion. Pavement configurations include granular with thin asphalt surface, deep strength asphalt and cement stabilised granular. Traffic loadings range from lightly trafficked local residential streets through industrial access roads to major arterial roads (Condric and Stephenson 2013). Of the sealed road network, almost 90% (4,952 km) has an asphalt surfacing, 9.8% a bituminous spray seal surfacing and 1.2% concrete or concrete pavers. The latter is almost exclusively confined to local residential streets (BCC 2011). A description of the traffic density category for Brisbane streets is presented in Table 3.2 (BCC 1993; BCC 2011).

Table 3.2: Original functional and traffic categories (BCC 1993; BCC 2011)

Functional Classification	**Pavement Type**	**Traffic Classification**	**Design Traffic (ESA) (Upper limit)**
Cul-de-sac and loop road	A	Local street	$1.5x10^4$
Collector road	B	Local street	$3.7x10^4$
Distributor road	C	Collector street	1.5×10^5
Sub Arterial road	D	Suburban road	7.5×10^5
Industrial road	E	Local road	1.5×10^6
Arterial road-minor	F	Arterial road	3.7×10^6
Arterial road- major	G	Arterial road	$> 1.0 \times 10^7$

Many of Brisbane's roads were neither designed nor built to modern standards and consist of many higher risk, lower quality pavements with a high degree of structural variability. Not only is there structural variability from street to street but also within streets. Many shoulder constructions have been constructed at a much later date than the centre pavement and not necessarily to a higher standard of construction. Due to the progressive development and rehabilitation of the network, it can be argued that there is a very fine balance between the structural adequacy of the pavements and the current loading. This suggests that parts of the local road network are particularly vulnerable to any reduction in strength caused by flooding (Condric and Stephenson 2013).

Brisbane City Council's 5,600 km road network sustained inundation during the extended high rainfall periods in the 2010-2011 summer wet season in Queensland. Between 600 to 1000 mm of rainfall was recorded in most of the Brisbane River Catchment during December 2010 and January 2011. Most of this rainfall fell between January 9 and 13, 2011 with rainfall exceeding the 1% annual exceedance probability (100 year Average Recurrence Interval) intensities for parts of the catchment basin (Condric and Stephenson 2013). This rainfall resulted in Brisbane experiencing its second highest flood since 1900 on January 13, 2011 (Honert and McAneney 2011). The Brisbane River was above the major flood level of 3.5 m above sea level (Australian Height Datum) at the City Gauge from January 12, 2011 to January 13, 2011, peaking at 4.46 m on January 13. Levels were above minor flood levels (1.7 m) from January 11 to 14, 2011. Approximately 22,000 homes and 7,600 businesses across 94 suburbs were flooded (BCC 2012).

Two years after the flood, the Brisbane City Council restored the city with work in excess of AU$400 million, including AU$127 million for roads and related infrastructure. Approximately 310 km of Brisbane City Council's road network was inundated due to flooding in January 2011 and approximately 145,659 square metres of pavements were resurfaced by the council (BCC 2012; BCC 2013). Table 3.3 shows the total network area as well as the approximate pavement area inundated for each traffic classification of roads in Brisbane. Overall, 4.9% of the pavements were inundated. The highest percentages of inundated pavements were for Arterial ("G") Roads (17.2%) and Industrial Access ("E") Roads (13%) (Condric and Stephenson 2013).

Table 3.3: The total network area and approximate pavement area inundated in Brisbane

Traffic Category		Total Network Area (m^2)	Approx. Flooded Area (m^2)	% Flooded Roads in Traffic Class
A	Cul-de-Sac/dead end	4,691,896	110,478	2.4%
B	Residential Collector	29,103,363	934,209	3.2%
C	Distributor	6,191,256	258,009	4.2%
D	Sub-arterial	6,243,718	286,780	4.6%
E	Industrial	3,538,454	459,640	13.0%
F	Arterial	1,480,970	105,236	7.1%
G	Major Arterial	2,695,503	464,939	17.2%
N	No Traffic	65,489	3,124	4.8%
Total		54,010,649	2,622,415	4.9%

3.3.1 Approach of Assessment by Condric and Stephenson (2013)

To identify the impact of flooding on the strength of the road network and its subsequent life, Falling Weight Deflectometer (FWD) testing was undertaken on a range of roads across flood affected areas of Brisbane. The sample of the network selected included a range of known pavement types with different traffic loadings. Pavement types included granular pavement with thin asphalt surface, deep strength asphalt pavement and cement stabilised pavement. Traffic loadings ranged from local residential streets through industrial access roads to arterial roads (Condric and Stephenson 2013). As BCC normally only undertakes FWD testing as part of the design process before a road is rehabilitated, there was limited data available to compare the post-flood deflections with pre-flood deflections for roads across the network. Two roads were identified where projects in the current resurfacing program had been tested prior to the flooding and were retested after the flooding (Condric and Stephenson 2013). Table 3.4 shows the list of roads selected for FWD testing after the flood.

Table 3.4: Post-January 2011 Flood-Road Strength Investigation

Suburb	Street	Length (m)	Extent of Inundation	Pavement Type	FWD Completed
Auchenflower	Haig Road	400	Partial	Thin AC & gravel	5/04/2011
Chelmer	Longman Terrace	212	Partial	Thin AC & gravel	25/02/2011
Chelmer	Luxford Street	133	Total	Thin AC & gravel	24/02/2011
Chelmer	Regatta Street	74	Partial	Thin AC & gravel	24/02/2011
Chelmer	Sutton Street	276	Partial	Thin AC & gravel	28/02/2011
Corinda	Deniven Street	541	Partial	Thin AC & gravel	11/04/2011
Graceville	Park Drive	227	Partial	80AC & gravel, 25AC, CTB	11/04/2011
Jindalee	Sinnamon Road	747	Partial	165AC / 100AC & gravel	11/04/2011
Milton	Haig Road	299	Total	50AC & 200CTB	5/04/2011
Mount Ommaney	Westlake Drive	1121	Partial	Thin AC & gravel	13/04/2011
Oxley	Aldersgate Street	380	Partial	Thin AC & gravel	13/04/2011
Rocklea	Franklin Street	337	Total	100AC & gravel	1/03/2011
Rocklea	Grindle Road	616	Total	100AC & gravel	5/04/2011
Rocklea	Sherwood Road	1979	Total	185AC / 200AC/210AC	20/02/2011
Sherwood	Sherwood Road	1017	Partial	200AC / 60AC & gravel	12/04/2011
South Brisbane	Cordelia Street	881	Partial	190AC / 60AC-130AC gravel	1/03/2011
St Lucia	Munro Street	609	Total	Thin AC & CTB or gravel	19/04/2011
Westlake	Westlake Drive	275	Partial	Thin AC & gravel	13/04/2011

A range of investigation strategies and analysis methodologies were used by Condric and Stephenson (2013). The initial assessment of structural adequacy was based on comparing the measured characteristic deflections against the tolerable deflection for the expected traffic loading for each road. To assess the magnitude of any structural deficiency, the overlay thickness needed to satisfy the design criteria to prevent subgrade rutting was calculated. These values were only calculated to show a comparative effect of flooding on reducing the pavement strength. Due to restrictions such as fixed levels of kerb and channel, overlays of these depths may not be practicable. Alternative treatments, such as stabilisation or total reconstruction in full depth asphalt, may be more appropriate (Condric and Stephenson 2013).

The principal failure criteria used to estimate remaining life was subgrade rutting using the design methodologies appropriate to the traffic loading level. Based on traffic loadings, time to reach this loading was determined. The design chart process employed by BCC is based on the Department of Main Roads (2007) approach. For heavily trafficked roads, the Department of Main Roads (2007) tolerable deflection chart (refer to Figure 3.2) (DMR 2007) was used to estimate remaining life in terms of ESAs. The overlay design chart for tolerable deflections follows Nomograph in Figure 3.2 (BCC 2011). The background and history of earlier pavement design practices and manuals are summarised in Nomograph. The TMR CBR 3% design curve is used for traffic loading ESA > 1×10^5. In traffic loading between 1×10^4 and 1×10^5 ESA, the tolerable deflections were adjusted to provide a smooth and continuous criterion (Nomograph). For traffic loading ESA < 1.0×10^5, caution should be exercised in applying the design chart provided in Nomograph since these values have not been fully validated to date (BCC 2011).

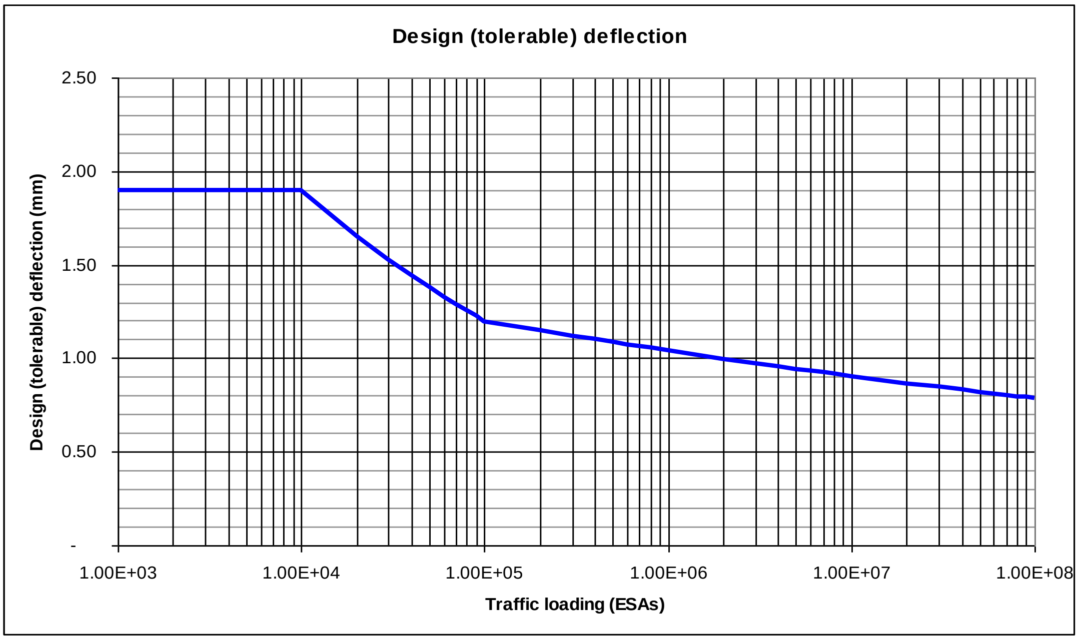


Figure 3.2: Nomograph for tolerable deflection on Council roads (BCC 2011)

An example is shown in Figure 3.3 for Park Drive, Graceville where before the flooding rutting of the subgrade did not control the life of the pavement. Post-flooding, rutting of the subgrade is estimated to occur in less than four years (Condric and Stephenson 2013).

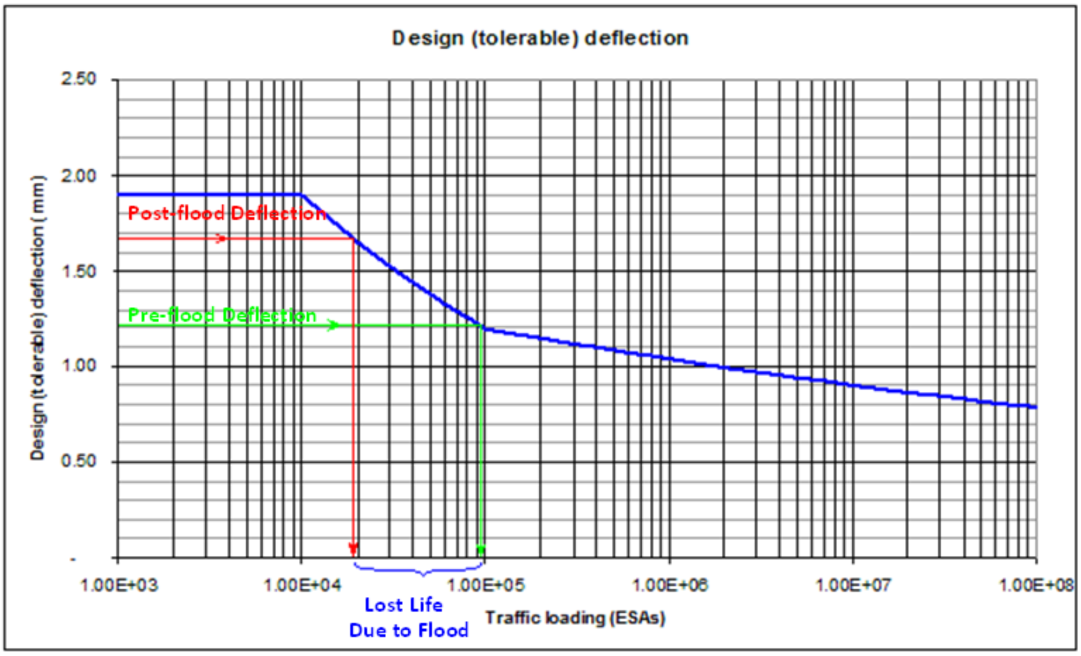


Figure 3.3: Park Drive, Graceville – effect of flooding on pavement life (Condric and Stephenson 2013)

For Lightly trafficked roads, the tolerable deflection chart from Council's "Pavement Rehabilitation Design Manual" (BCC 2011) was used to estimate remaining life. The loading in terms of ESAs was converted to years based on the traffic loading. For example, for Park Drive (Graceville), the pre-flood life was approximately 40 years compared to a post-flood life of approximately 10 years (Condric and Stephenson 2013).

The 1993 BCC Pavement Rehabilitation Design Guide (BCC 1993) applied Design tolerable Deflection, $D_{tol} = 2.1$ mm for traffic loading up to 3.7×10^4, and $D_{tol} = 1.6$ mm for traffic loading up to 1.5×10^5. These values were derived from field investigation, and based on the assumption, that 'a representative rebound deflection value is the mean of adjusted measured rebound deflections plus two standard deviations'. Accepting that the coefficient of variation (CV) was 0.5 in these calculations, 0.2 mm tolerable deflection reduction can be applied if the representative deflection will be calculated as the mean deflection plus 1.64 standard deviation ($\bar{x}+1.64s$). This adjustment is necessary since the tolerable deflection for traffic loading of ESA < 1.0×10^5 was considered by City Design as being too high. Tolerable deflection values used in the last four decades are summarized in Figure 3.4 - BCC Tolerable Deflection Curves (1993 to 2010) (BCC 2011).

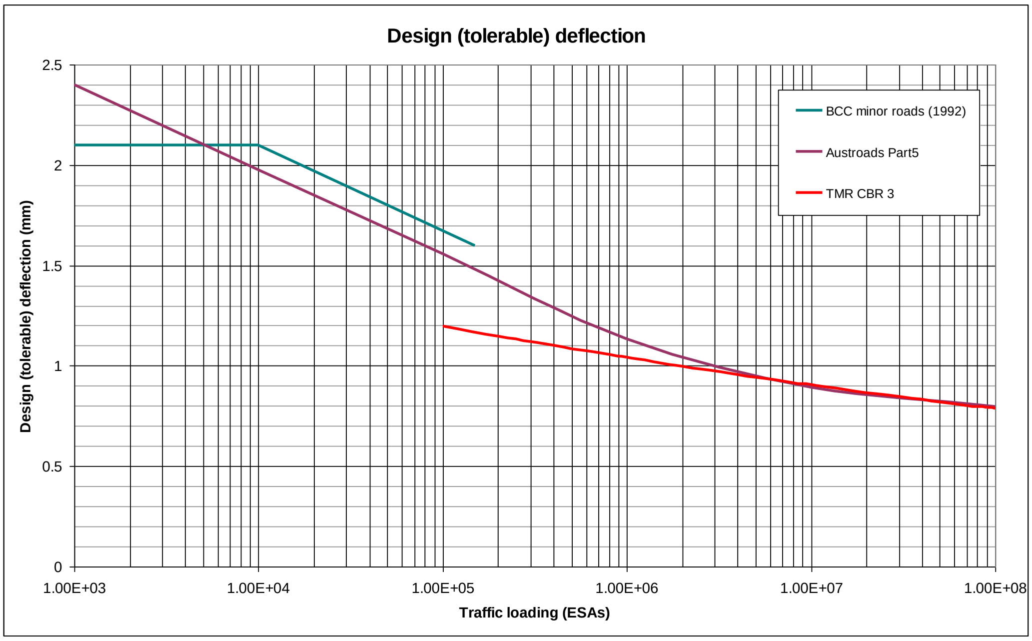


Figure 3.4: BCC Tolerable Deflection Curves (1993-2010) (BCC 2011)

Table 3.5 shows the streets with pre-flood FWD data and their remaining life span. The resultant asset damage by the flooding, and consequent loss of life for both the new and older streets required significant early intervention (Condric and Stephenson 2013).

Table 3.5: Estimated loss life of streets with pre-existing FWD testing data (Condric and Stephenson 2013)

Street/ Road	Suburb	Length (m)	AC Depth (mm)	Pavement Type	Traffic Density (No. of ESAs)	Pavement Age	Remaining Life (years)		Lost Life (years)
							Pre-Flood	Post-Flood	
Munro Street	St. Lucia	151	25	Gravel	$1.5x10^4$	11	12.5	5.7	6.8
Luxford Street	Chelmer	133	50	Gravel	$3.7x10^4$	36	2.6	0	2.6
Luxford Street	Chelmer	133	50	Gravel	$3.7x10^5$	36	7.5	2.1	5.4
Park Drive	Graceville	112	25	230 CTB	$4.1x10^4$	15	>40	32.8	8
Haig Road	Milton	120	50	200 CTB	$1.4x10^6$	22	>40	>40	0
Haig Road	Milton	179	50	200 CTB	$1.4x10^6$	22	>40	>40	0
Haig Road	Milton	120	50	200 CTB	$1.4x10^6$	22	36	2	34
Haig Road	Milton	179	50	200 CTB	$1.4x10^6$	22	>40	>40	0
ESA= Equivalent Single Axle									

Condric and Stephenson (2013) indicated a significant reduction in pavement strength due to the ingress of water, which damaged and weakened supporting subgrade layers, after the January 2011 flooding. Visual inspections showed that extensive areas of surface and pavement failures occurred. Accelerated deterioration and loss of pavement life, due to the inundation and prolonged wet weather, were identified.

The analysis showed that there has been significant reduction in the total load carrying capacity of the pavement which was expressed in terms of reduced life. Reductions in estimated life of up to 75% were identified. The study showed that the strength of the pavements was significantly reduced by flooding due to weakening of the supporting subgrade and/or granular layers. For thick asphalt pavements, the inundation has had limited impact on the strength of the upper asphalt layers. The analysis considered the pavement in potentially its weakest condition following flooding. It could be anticipated that subgrade and pavement material may eventually dry out and increase in strength. Therefore, the estimated reductions in life based on FWD test results are the absolute worst case scenario. How long the pavements remain in this weakened condition is a matter of some conjecture. However, any damage sustained due to loading the pavement whilst in these weakened conditions will remain after the pavements have dried out (Condric and Stephenson 2013).

Condric and Stephenson (2013) indicated that the key pavement performance indicators of maximum deflection and curvature function were significantly different between the non-flooded and flooded sections. In most cases, the flooded pavements needed to strengthen to reach their normal design lives while the non-flooded pavements generally had sufficient strength to meet their design lives. In the cases of Longman Terrace (Chelmer) and Regatta Street (Chelmer), which were partially inundated, there are higher deflections in areas that were not inundated. The pavement surface in these areas had extensive cracking which may have allowed rain water entry. These areas had been previously identified for reconstruction in the pavement design reports. Overall, the subgrade response is lower for the flooded sections indicating that the reduction in pavement capacity is due to loss of strength (reduced CBR value) in the supporting subgrade (Condric and Stephenson 2013).

Franklin Street (Rocklea) and Sutton Street (Chelmer) had deflection ratios (D_{250}/D_0) below 0.5 indicating the presence of low strength granular materials that were impacted by flooding. This is more critical for the granular pavements in the industrial areas, such as Franklin Street (Rocklea). In Park Drive (Chelmer), the Deflection Ratios were similar for both the inundated and non-inundated sections indicating that the pavement gravels were not adversely affected. For Deniven Street (Corinda), the deflection ratios of the flooded sections were 25% to 35% higher than those of the non-flooded sections. The relatively low deflection ratios for the non-flooded areas indicate a poor quality granular pavement. On Westlake Drive (Westlake) the Deflection Ratios for the non-flooded sections of 0.30 to 0.44 suggest a poor quality granular pavement, whereas in the flooded sections they were 0.51 to 0.56. Normally, higher deflection ratios are associated with higher quality pavements. For granular pavements in a saturated state, the relatively rapid loading of the FWD may not allow the pore water pressure in the granular pavement to dissipate thus indicating a higher strength pavement than actually exists (Condric and Stephenson 2013).

The Curvature Function (D_0 - D_{200}) is an indication of the stiffness of the upper layers of the pavement. In the deep strength asphalt pavements of Cordelia Street (South Brisbane) and Sherwood Road (Rocklea), the Curvature Functions for both the non-flooded and flooded pavements were similar. This shows that the upper asphalt layers were not affected by the flooding and supports Council's practice of using deep asphalt layers (Condric and Stephenson 2013).

3.4 STUDIES ON TMR QUEENSLAND ROADS

Khan et al. (2014a), (2014b) and (2015) are the only available studies for Australian road condition which attempted to incorporate flooding events in the development of probabilistic models. Although probabilistic models are outside the scope of this research, the above mentioned studies are worth discussing as a part of the extensive literature review and to review the method adopted in this research and methods adopted by other studies. Some terms related to probabilistic models is included in Appendix A.

The intensity of flooding and water ponding varies storm to storm, and the frequency of flooding is also uncertain. To date, no road deterioration model can directly address a flooding event in a pavement's life cycle performance (Khan et al. 2015). Moreover, there is no cost-effective maintenance strategy to select an appropriate rehabilitation treatment as a post-flood strategy. Khan et al. (2014a, 2014b) suggested that a probabilistic road deterioration model incorporating uncertainty of events, such as flooding, should be used for asset management purposes. Moreover, the derivation of post-flood and optimum maintenance and rehabilitation (M&R) strategies should also be incorporated into pavement life-cycle analysis. Khan et al. (2014b) aimed to derive roughness-based and rutting-based road deterioration models that considered flooding and pre- and post-flood strategies. The study covered flood-damaged pavements that were saturated but the embankment and structures were intact (not completely damaged or washed away); i.e. roads that are at moderate risk and need rehabilitation after flooding. In fact, these roads need proper attention after a flooding event (Khan et al. 2014b).

As a case study, the study considered flooding in Queensland, Australia, and used 34,000 km of the Queensland's main road authority's road database. The study used the HDM-4 model for developing post-flood and optimum maintenance and rehabilitation (M&R) strategies with flooding considering pavement life-cycle analysis. The study used IRI and rutting data for modelling (Khan et al. 2014a). The approach used to derive models with flooding by Khan et al. (2014b) is shown in Figure 3.5.

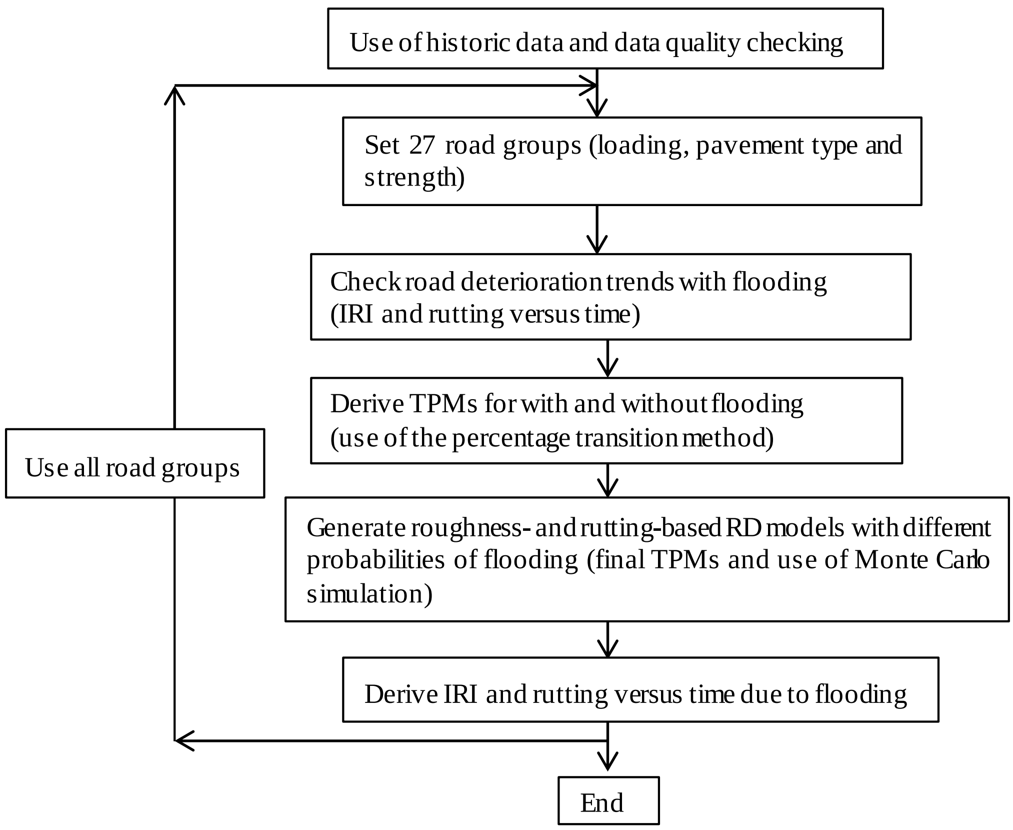


Figure 3.5: Approach to deriving RD models with flooding (Khan et al. 2014b)

The study developed 27 road groups based on pavement type, traffic loading and pavement strength. The road grouping considered three types of pavement – flexible, composite and rigid. Similarly, traffic loading was divided into three types (high, medium and low) and pavement strength into three categories (strong, fair and poor). The new specific road deterioration model was valid for a specific road group and, as a result, all of the models were suitable for the whole road network (Khan et al. 2014b).

Two road groups out of 27 demonstrated the numerical results of an RD model incorporating flooding. The two road groups were a flexible pavement with low traffic loading and high strength (F-LT-S) and a flexible pavement with low traffic loading and poor strength (F-LT-P). Deterioration models for the two road groups revealed that if the first year impact was considered then F-LT-S performs better than F-LT-P. However, this was not valid for the remaining six years where the stronger pavement showed less deterioration, up to 10% probability of flooding, but beyond that it performed less well. The stronger road performed better at different flooding probabilities when rutting-based RD models were compared (Khan et al. 2014b).

A new approach to the development of a post-flood maintenance strategy was proposed in Khan et al. (2015). Previously, normal deterioration was used to select a post-flood rehabilitation, which may not be necessarily suitable. This study used the newly derived roughness and rutting-based RD models to predict deterioration in years 2, 3 or 4 assuming a flood in year 1. As a result, it was possible to obtain appropriate treatments. The HDM-4 model was used to derive a post-flood strategy assuming a flood in year 1 and post-flood rehabilitation starting from year 2. Three different years were chosen, as it is difficult to rehabilitate a whole network given the time required to design post-flood rehabilitation, procurement processes and allocate funding, which is also the case for the TMR-QLD. After rehabilitation, normal deterioration was assumed in the remaining life cycle analysis. Therefore, only one flood event was assumed in a life cycle (Khan et al. 2015).

Different optimisation objectives have different aims to achieve; as a result, they provide different solutions for a road group. Considering the key factors, i.e. agency costs, budget, economic benefits, life cycle performance and suggested treatments, the 'minimise agency cost at target IRI' optimisation objective results may be chosen as the final solution. A road authority may assess all the flood-damaged roads before implementing any of the strategies, basically solutions with constrained and unconstrained budgets. A road authority has to properly investigate its flood-damaged roads prior to any implementation. If the budget is unconstrained, the results obtained with the 'maximise ΔIRI' could be chosen, which can keep the average road network at 2.0 IRI in its life cycle. This study assumed a flood at year 1 in the life cycle. If this flood comes in any other year, then additional maintenance costs would be required at AU$0.9 billion per year from year 1 to the before flood year. Moreover, if two or more flood events occur, then separate analysis is needed to predict after-flood road deterioration using the deterioration models at the probability of flooding. Finally, HDM-4 analysis would identify different post-flood strategies with extra costs (Khan et al. 2015).

3.5 DETERIORATION MODELLING FOR EXTREME WEATHER EVENTS

While many studies have attempted to capture the effect of the environment, load and pavement structure on pavement failures, only a few have realised the impact of severe extreme weather events, such as snow storms and floods, on road infrastructure. Tari et al. (2015) and Shamsabadi et al. (2014) developed pavement deterioration models considering extreme weather events. These impacts were first quantified using the Long Term Pavement Performance (LTPP) and National Oceanic and Atmospheric Administration (NOAA) databases with a dependable natural deterioration model. Then, a regression-based statistical approach was undertaken to model the effect of snow storms and floods on pavement serviceablilites based on the severity of the events and condition of the pavement prior to the events. Final models rendered more than 90% correlation with the quantified impact values of snow storms and floods (Shamsabadi et al. 2014).

Second, the effect of extreme events on road deterioration was quantified by forward-projecting IRI values from one measured IRI to the point where the event occurred. Consequently, they backward projected the next measured IRI to the point where the event occurred; the difference between these two values is due to the extreme event that happened in that month if no other events/maintenance activities had taken place in that period. This is illustrated in Figure 3.6. To fit a model to these quantities, they had to first collect the data needed (Shamsabadi et al. 2014).

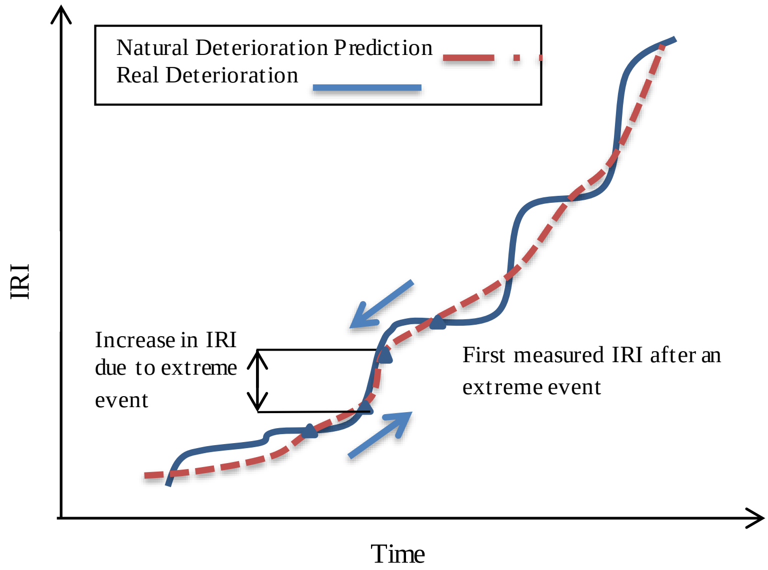


Figure 3.6: Quantifying the increase in IRI due to extreme events (Shamsabadi et al. 2014)

Eight parameters were entered into a stepwise regression model for 28 sections affected by a single flood, the remaining seven sections were used for testing (Shamsabadi et al. 2014). The final Equation derived through the fusion model was:

$$\%\Delta IRI = 10.7 - 1.66NIRI + 7.30NDepth - 2.10NDuration + 14.3Depth \times IRI \quad (3.1)$$

where,

%ΔIRI	=	Percentage increase in IRI due to the snow storm
NIRI	=	Normalised IRI of the section before the snow storm
NDepth	=	Normalised Depth of the snow storm
NDuration	=	Normalised Duration of the snow storm
ESAL	=	Equivalent Single Axle Load (derived from traffic)

In the study of Tari et al. (2015), performance data from LTPP and extreme weather events data from NOAA were not from the same month. Quantifying the impact of extreme weather events needs prediction of performance data for the month in which they occurred. The natural deterioration model proposed by Jackson and Puccinelli. (2006) (refer to Equation (3.2)) was used to make this prediction. Immediate IRI recorded before an extreme weather event was forward-projected to the time of the event by the natural deterioration model. Then, the immediate IRI value recorded after the event was backward-projected to the time of the event. The difference between the forward-projected and the backward-projected IRI values is the additional deterioration caused by the extreme event occurring in between (Tari et al. 2015).

$$Ln(\Delta IRI + 1) = \begin{array}{l} Age(4.5FI + 1.78CI + 1.09FTC + 2.4PRECIP \\ + 5.39Log(ESAL) / SN) \end{array} \quad (3.2)$$

where,

ΔIRI	=	Change in International Roughness Index
Age	=	Pavement Age
FI	=	Freezing Index (Degree-days when air temperatures are below and above zero degrees Celsius)
CI	=	Cooling Index (Temperature relation to the relative humidity and discomfort)
FTC	=	Freeze-thaw Cycles
PRECIP	=	Precipitation
ESAL	=	Equivalent Single Axle Load (Conversion of traffic into single axle load)
SN	=	Structural Number

Stepwise regression was implemented separately on two sets of the acquired data: sections impacted by Floods and sections impacted by Snow Storms. For Snow Storms, 29 out of the 42 available sections were entered into the fusion model for training at the confident interval of 95%, producing the model in Equation (3.3).

$$\%\Delta IRI\_S = \begin{array}{l} 5.09 - 2.5NIRI + 1.7NDepth - 1.74NDuration + 0.76ESAL \\ \times Duration \end{array} \quad (3.3)$$

Equation (3.4) was rendered after training on 28 sections affected by a single flood.

$$\%\Delta IRI\_F = \begin{array}{l} -4.47 + 0.48IRI - 0.23NDepth - 0.57NDuration - \\ 26.49ESAL -0.49Depth \times IRI \end{array} \quad (3.4)$$

The final deterioration model which considers both natural causes and occurrences of floods and snow storms is provided in Equation (3.4).

$$\Delta IRI = \Delta IRI\_N + \Delta IRI\_F + \Delta IRI\_S \tag{3.5}$$

where:

$\%\Delta IRI\_S$	=	Percentage increase in IRI due to the snow storm
IRI	=	Normalised IRI of the section before the snow storm
NDepth	=	Normalised Depth of the snow storm
NDuration	=	Normalised Duration of the snow storm
ESAL	=	Equivalent Single Axle Load (Conversion of traffic into single axle load)
ΔIRI	=	Overall increase in IRI
ΔIRI_N	=	Increase in IRI due to natural causes (Equation (3.2))
ΔIRI_S	=	Increase in IRI due to a single snow storm (Equation (3.3))
ΔIRI_F	=	Increase in IRI due to a single flood (Equation (3.4))

3.6 STUDIES ON THE MISSOURI RIVER FLOODING BY VENNAPUSA ET AL. (2013, 2016)

In 2011, the Missouri River flooding caused significant damage to many geo-infrastructure systems including levees, bridge abutments/foundations, paved and unpaved roadways, culverts, and embankment slopes along the Missouri River basin extending from Montana to Missouri. The total reported direct cost to repair flood damage to the transportation infrastructure on primary and secondary roadways in these counties was about US$63.5 million. The extent of damage was in some cases directly observable, i.e. where segments of the roadway were washed away, but in many cases was undetermined, i.e. where the damage was below the pavement surface or around bridges. The main goals of this research project were to assist county and city engineers by deploying and using advanced technologies to rapidly assess the damage to geo-infrastructure and develop guidance for repair and mitigation strategies and solutions for use during future flood events in Iowa. In situ testing involved conducting FWD, DCP, and ground-penetrating radar (GPR) testing, three-dimensional (3D) laser scanning, and hand auger soil borings. In situ testing was conducted on about 30 km (18.6 miles) of roadway, where the test segments varied in length from about 150 m (500 ft) to 7.0 km (4.3 miles). The test segments varied by flood condition (fully or partially flooded), and type of surfacing [gravel, chip seal surface over stabilised or unstabilised gravel base, Portland cement concrete (PCC), and Hot Mix Asphalt (HMA)] (Vennapusa et al. 2013).

Although Vennapusa et al. (2013), (2016) discussed the damage observed for all geo-infrastructures, this literature reviews only discusses the damage to paved roadways. Based on field reconnaissance of the flood-damaged areas, review of the damage inspection reports submitted to the Iowa Department of Transportation, and interviews with county engineers, the damage observed on paved roadways were as follows (Vennapusa et al. 2013, 2016):

- Voids at shallow depths [< 150 mm (6 in.)] due to erosion of underlying base material
- Voids at deeper depths [> 150 mm (6 in.)] due to erosion of subsurface material
- Partial to complete erosion of PCC and HMA pavements and underlying base material
- Erosion of granular shoulders

FWD tests obtained shortly (< 30 days) after flooding indicated that the average modulus values in flooded zones were about 1.3 to 3.6 times lower than the values in non-flooded zones. In some areas, the foundation layers within the flooded zone gained strength over time, likely as the degree of saturation in the subgrade decreased. However, many sections did not show much improvement. FWD surface modulus measurements were influenced more so by the subgrade layer (which was relatively weaker) than the surface gravel layer (Vennapusa et al. 2016).

3.7 VULNERABILITY ASSESSMENT OF ASPHALT PAVEMENTS BY MALLICK ET AL. (2014)

Mallick et al. (2014) presented a rational procedure to assess the vulnerability of asphalt pavements to flood induced damage. A system dynamics based methodology was developed to determine the critical time ($T_{critical}$) for full saturation of the unbound base and for failure of the bound surface layer. The methodology and the web-based simulation tool presented would help users to identify potentially vulnerable stretches of highway prior to flooding and either take action to improve them or monitor them closely to obtain pre-flood conditions which can be compared against post-flood conditions to detect deterioration. It would help them decide whether emergency and non-emergency vehicles could be allowed during and immediately after flooding, and in planning post-flooding investigative actions (Mallick et al. 2014).

In most parts of the world, the majority of the road pavements are built with Hot Mix Asphalt (HMA), with unbound aggregate base underneath the HMA layer. Moisture in either the HMA layer, and/or in one or more of the underlying granular layers may damage HMA pavements that are subjected to flooding. In this study, a framework was developed to predict the likelihood of damage of an asphalt pavement under flooding for a specific water depth and flood duration; i.e. what would be the critical time period ($T_{critical}$) such that if the time of flooding > $T_{critical}$, the pavement could be predicted to be severely damaged during and immediately after flooding. The damage is defined in terms of saturation – for periods of flooding > $T_{critical}$, the entire thickness of the base course under the Hot Mix Asphalt (HMA) layer is considered to be completely saturated and damaged (Mallick et al. 2014). As this study is not highly relevant to this literature review and study, a detailed review was not included in this section.

3.8 PAVEMENT STABILISATION TECHNIQUES TO RESTORE FLOOD-AFFECTED ROADS

The study by Lee et al. (2014) presented the findings from field and laboratory investigations undertaken on the flood-affected pavements of four regional roads near Ipswich in South East Queensland. These investigations were undertaken to characterize existing pavement materials and to evaluate the performance of existing granular materials stabilised with foamed bitumen or modified with cementitious stabilising agents. To evaluate the mixes of granular materials and stabilising agents, the results from tests conducted on samples of existing / imported granular materials stabilised or modified with foamed bitumen or cementitious binders were compared against a set of Australian developed mix design criteria. Furthermore, this study summarised the advantages and disadvantages of both the cementitious modification and foamed bitumen stabilisation techniques and summarised the manners in which these layers were mechanistically modelled.

As part of the road restoration project, the field assessment of the pavement damaged areas in the Metropolitan and Darling Downs Region of the TMR was conducted. The process also included the identification of pavement failure mechanisms, the execution of field and laboratory tests, and the design of a number of pavement rehabilitation options.

This study presented the findings from the field and laboratory pavement investigations for roads damaged by rainfall / flood events on four regional roads within Ipswich in South East Queensland. The four regional roads fall within the jurisdiction of the Metropolitan Region of the TMR, Queensland (Lee et al. 2014).

Most affected pavements consisted of granular layers with multiple sprayed seals over soft, and often highly reactive natural subgrade. Soft natural soils that often exhibit high swelling potential coupled with poor road geometry (boxed-in pavements) and poor surface and sub-surface drainage (poorly maintained table drains and non-existent sub-surface drains) often lead to the granular pavements and the subgrade having large seasonal water content fluctuations. These large moisture fluctuations lead to severe pavement failures. Foamed bitumen stabilisation and cement modification were two commonly used pavement treatment methods in the project as they allowed the re-use of granular materials of the existing pavement by increasing their strength and reducing their susceptibility to water ingress (Lee et al. 2014).

3.9 MANAGING FLOOD AFFECTED PAVEMENTS

Road pavements are designed to match the prevailing conditions and available local materials across Queensland (TMR 2014a). When a road is inundated, especially for a long time, the materials in the road pavement become saturated. This reduces the strength of the road pavement material. The department undertakes inspections and testing to re-open roads as soon as possible after inundation. In its custodial role, the department balances protection of the road asset with community and industry access. Roads that have been saturated must be carefully managed. To do this, road closures, load restrictions and traffic management are the only options available (TMR 2014a).

Once road pavements are saturated, it takes time for moisture to drain from the material and for full-strength properties to return. Restricting the loads on roads during this ‘dry-back’ period is essential to preserving this important community asset. The responsible departmental Regional/District Director will progressively lift load restrictions according to the local conditions. The stages include (TMR 2014a):

- 5 tonne limit to allow access for local residents and emergency services vehicles
- 80% of the legal axle loads on transport vehicles as shown in Table 3.6
- removal of load restrictions.

Table 3.6: Load Restrictions

Axle Group	Legal Load	80% Load
Steer axle	varies	no reduction
Single 4 tyre	9.0 t	7.2 t
Tandem 8 tyre	16.5 t	13.2 t
Tri 12 tyre	20.0 t	16.0 t

In making these decisions, the Regional/District Director relies on specialist advice based on measurement and analysis of road surface movements under test loading of the pavement. This technique models the 'dry-back' process of the road pavement material, and allows the decision to be made about when it is safe to remove the load restrictions and open the road to all road users.

While flood waters cover the road, road closures are essential for road user safety and to avoid damage to roads and bridges. Sometimes road closures are extended after the water recedes. This can be due to:

- sections being eroded or washed-out
- flood debris on roads and bridges
- damage to bridges and culverts which may compromise road user safety.

At all times, safety is the number one priority. The department will endeavour to minimise the duration of road closures. However, the length of a closure depends on the time needed to:

- inspect and assess the roads and bridges to ensure they are safe
- remove flood debris
- make the necessary repairs or build temporary sidetracks (TMR 2014a)

3.10 SUMMARY

This chapter reviewed and discussed literature which assessed and evaluated the impact of flooding on road pavement systems. Although a few studies have attempted to incorporate flooding events on pavement, no specific correlation has been developed to indicate the rate of pavement deterioration using field data of flood affected pavements.

This highlights the necessity for further investigation of flood-affected pavements and justifies the need for development of models, especially for flood affected pavements.

While modelling deterioration of pavement under flooding conditions, it is necessary to have data before and after floods. If the inundated pavements are monitored annually, it will be easier to understand the behaviour of pavement after the flood. This is key to knowing whether a pavement is gaining strength during subsequent dry periods or needs rehabilitation. Modelling will also help analyse the remaining life cycle of pavements. For this reason, long term observation of flood affected pavements is necessary.

4. RESEARCH METHODOLOGY AND DATA COLLECTION

4.1 RESEARCH METHODOLOGY

A research project should be systematic, logical, empirical and replicable. It should be structured, with specific steps to be taken in a specified sequence in accordance with a well-defined set of rules. A systematic research design specifies the objectives of the study, methodology and techniques to be adopted to achieve the objectives (Shajahan 2005). Methodology guides the researcher to collect data, analyse and interpret findings. This research has followed a structured methodology to achieve the research objectives and answer the research questions. This chapter presents the research methodology, justification, data collection processes, and sources of data.

This study commenced in early 2013 and was funded by Austroads and the ARRB Group Ltd in a collaborative research arrangement between ARRB and Griffith University. This research was conducted in three stages. At the preliminary stage, the study focussed on the collection and examination of the structural and surface condition data of flood-affected pavements. The aim and scope of the research was identified and initial data on flood affected pavements across Queensland were collected at this stage. This research used the Falling Weight Deflectometer (FWD) deflection and surface condition (roughness, rutting and cracking) data sourced from Brisbane City Council (BCC), Roads and Maritime Services, New South Wales (RMS, NSW) and Department of Transport and Main Roads (TMR), Queensland. In stage two, yearly data of flood affected pavements were collected in detail and pre- and post-flood data were compared. Stage three involved data analysis, selection of pavement sections for modelling and development of deterministic models for deterioration of structural and surface condition (roughness and rutting). It should be noted that the surface condition parameter ‘cracking’ was eliminated from the analysis due to insufficient evidence of the impact of floods on accelerating the rate of cracking.

The first step of the research was to identify the problem; set the research aim and questions and identify the research knowledge gap, which were discussed in Chapter 1.

An extensive review of the literature was conducted and presented in Chapters 2 and 3. This review partly achieves the aim of the research and answers Research Question 1 which is “What are the necessities of specific pavement deterioration models for roads subjected to flooding?” The literature review on existing pavement deterioration models presented in Chapter 2 and literature review on flood affected pavements in Chapter 3 examine if any current models could predict the deterioration trends for flood affected pavements. As there was no specific deterioration model available for flood affected pavements, it was imperative to develop models to accurately predict the deterioration of these pavements.

Chapters 5 and 6 provided answers to Research Questions 2 and 3 by collecting data on flood affected pavements and conducting extensive analyses (What is the rate and trend of pavement deterioration before and after inundation? What are the long term consequences of flooding on the inundated pavements?). These two chapters also include the new models developed for deterioration of structural condition and surface condition, such as rutting and roughness. MS Excel, SPSS (SPSS Version 22.0 2013) and CIRCLY5.0 (Wardle 2009) were used for data processing and analysis. SPSS was used for development of the models.

It was essential to obtain strength data measured by FWD testing for roads submerged by gradually rising flood water. Consequently, this research considered the following types of flood affected roads:

- Pavements flooded by gradual rise of flood water
- Pavements that did not endure any catastrophic damage, such as being washed away by flood.
- Pavements that were rehabilitated after flooding due to the damage sustained from the flooding events.

The research did not include roads completely washed out or damaged by flood water as damage of such kind is catastrophic and needs immediate action. It was also very important to conduct analyses on the pavement sections that have strength and surface condition data available for the same road section pre- and post-flood.

Development of a pavement deterioration prediction model is a tedious task and must reflect a good data set. Pavement inventory data such as pavement composition (base, sub base, subgrades thickness), type, age, maintenance history/records, and rehabilitation history were collected for the assessment and development of modelling for the deterioration of the structural and surface condition (rutting and roughness) of flood affected pavements. Figure 4.1 shows the general methodology of the data analysis.

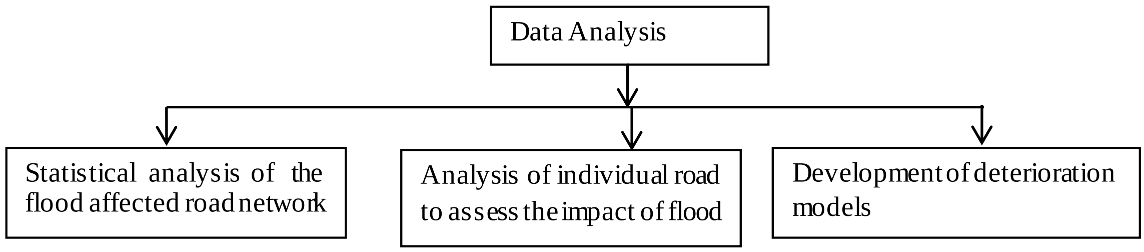


Figure 4.1: Summary schematic of data collection and analysis

In general, data analysis was conducted in three steps which are as follows:

- Assessment of the flood affected road network using various statistical analysis techniques
- In depth analysis of individual roads to assess the impacts of flood
- In depth analysis of the selected pavement sections for the development of pavement deterioration models.

The organisation and analysis techniques of structural condition data is shown in Figure 4.2. The organisation and analysis techniques of rutting and roughness data is shown in Figure 4.3. MS Excel was used to plot data and SPSS was used for analysis, comparison of data and to develop the models.

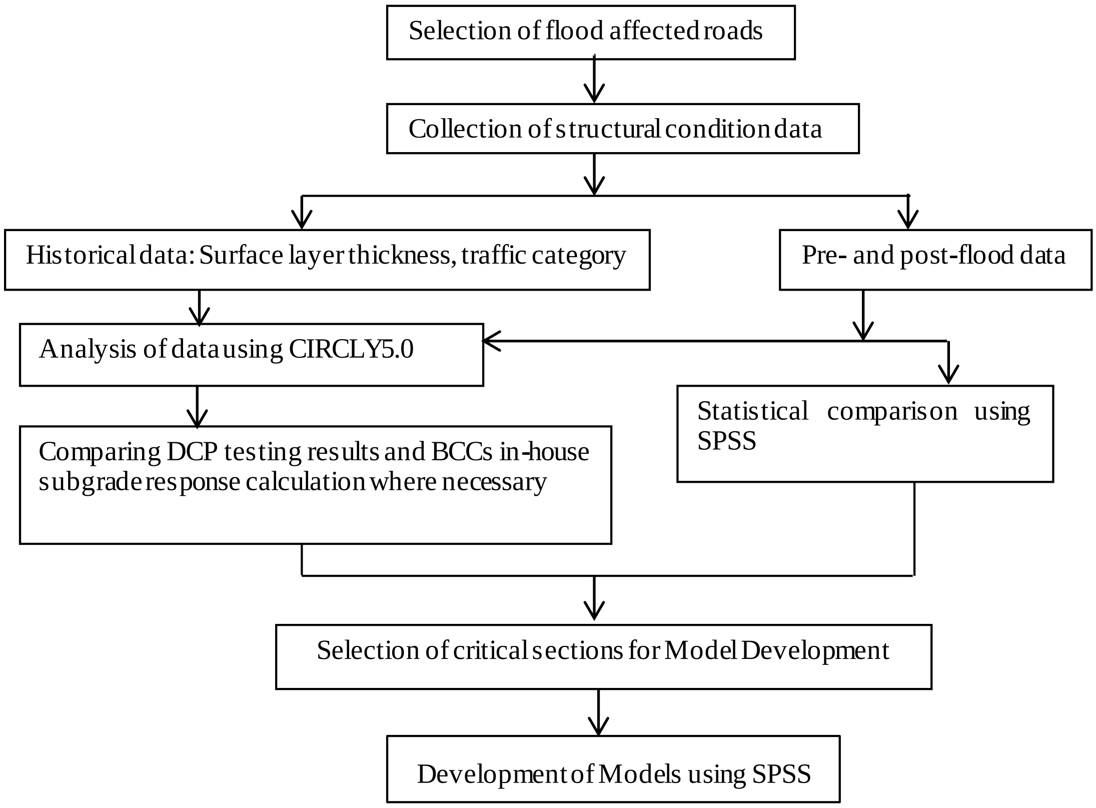


Figure 4.2: Summary schematic of structural condition data collection, analysis and development of deterioration model

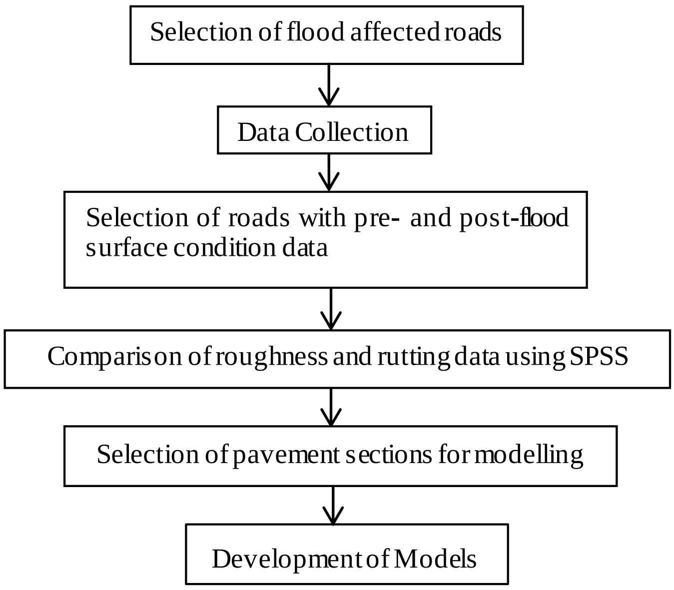


Figure 4.3: Summary schematic of data collection, analysis and development of models for rutting and roughness

In summary, this study analysed data, assessed the impact of floods on pavement deterioration and developed mechanistic-empirical, deterministic-based deterioration models to predict rapid deterioration of flood affected pavements.

4.2 PRINCIPLES OF SAMPLING AND MEASUREMENT OF DATA

It is important to properly collect and sample data for project level analysis of pavements. The selection of number of samples and spacing length of pavement network depends upon the aim of the sampling process. Austroads (2003) recommends that if the aim of sampling is to detect local weaknesses in pavement strength, such as that needed for a project level investigation, then the interval of strength measurement sampling needs to be set to detect variations in strength over relatively short lengths of pavement, say less than 20 m. Project aims that involve sampling long lengths of pavement along road links in a network are required to (Austroads 2003):

- establish an assessment of the representative structural condition of the pavement for strategic analysis (budgets and strategic maintenance and rehabilitation intervention) and contractual performance purposes; and/or
- detect significant lengths of pavement (segments > 100 m) whose sampled assessment of representative strength appears to be approaching a value where further detailed structural investigation and structural intervention may be warranted.

A sampling interval of 50 to 100 m uniform spacing for strength measurement over several kilometres of a defined road link can be undertaken within the network as a trial in the first year of network strength assessment (Austroads 2003). For two lane roads, sampling for strength testing (using all testing devices) is undertaken in the inner and outer wheel paths of both lanes or a single lane to assess seasonal effects if they are considered relevant. Seasonal effects are particularly relevant where extremes of climate, such as wet and dry, are experienced annually. Seasonal corrections can be made to outer wheel path deflections to estimate the deflections that would occur at the 'end of wet season' condition where it is not possible to measure deflections at this time. It may be appropriate to test the samples in the wheel paths of only one lane where its traffic levels are known to be significantly higher than the adjacent lane.

With Falling Weight Deflectometer (FWD) testing on two lane roads, strength sampling is typically undertaken in the outer wheel paths at the same spacing between consecutive strength measurements along each of the two lanes. The assessment of seasonal effects, if relevant, requires FWD testing in both wheel paths. In general, any sampling process must be able to sample potential significant differences in pavement/subgrade strength along defined road links. This will allow these links to be sectioned, or segmented, into relatively ‘homogenous’ sections for traffic loading, climate and condition (accounting for both the surface and structural state of the pavement). This requirement for ‘homogenous’ sections should be the basis for developing an appropriate sampling process. Continuous sampling is obviously the ideal (Austroads 2003). As this study collected data from Brisbane City Council, TMR, Queensland and RMS, NSW, the above mentioned principles of sampling and measurement were followed by the relevant organisations.

4.3 JUSTIFICATION FOR DATA COLLECTION

The success of a PMS is highly dependent on the accurate prediction of future condition of pavement sections. Based on the pavement condition predictions of a deterioration model, a PMS provides future estimations for the planning of maintenance and rehabilitation activities. If the PMS is to project forward conditions and costs with any degree of accuracy, then the data that is used to calibrate the pavement deterioration models must be as accurate and appropriate as possible, and collected over a reasonable period of time (Chai et al. 2010).

A sound pavement evaluation will enable the designer to assess the existing pavement and determine its current condition, identify the causes/mechanisms of any observed pavement distress, ascertain whether the existing pavement must be rehabilitated to withstand the predicted conditions for the required design period; and provide foundation for identifying the required treatments/interventions (TMR 2012). TMR (2012) recommends investigations of the factors affecting the performance of the existing pavement to determine its functional condition (e.g. how the road satisfies the needs of road users in terms of cost, comfort, convenience and safety) and structural condition (i.e. how it responds to load[s]). Pavements’ ability, in terms of its functional and structural capacity, to withstand the traffic and other environmental conditions expected over the required design period are also determined in this process. If rehabilitation measures are necessary, the evaluation will also allow these to be designed so that the required life is attained. Premature conclusions about the causes of pavement distress should be avoided as they may cause the designer to focus on justifying them, rather than being open to all possibilities. As a result, the designer may fail to identify the real cause(s) of the pavement distress (TMR 2012).

Information used in a pavement evaluation is available from a number of different sources. These include historical records, pavement condition assessments, an assessment of the drainage systems, including any sub-surface pavement drainage systems, materials testing and the pavement's structural response to load (TMR 2012). TMR (2012) emphasised that a data map should consist of a schematic plan of the pavement under evaluation, on which some or all of the following detail may be symbolically, diagrammatically or graphically represented:

- Location, nature, extent and severity of defects including patches
- Load response (e.g. deflections as recorded by FWD or similar type of testing methods) against chainage
- Identification of representative sections as determined by condition data (e.g. by using deflection results)
- Values that characterise representative sections (e.g. characteristic moduli derived from back analysis)
- Test results (e.g. of subgrade California Bearing Ratio [CBR])
- Changes in pavement or subgrade configuration or type
- Details and extents of type cross sections
- Geometric features (e.g. longitudinal sections, superelevation/crossfall, alignment)
- Drainage features
- Topographical features (e.g. cuts, fills, cross grades)
- Degrees of saturation
- Moisture contents
- Photographs (e.g. from field inspection).

The benefits of such a map include assistance in identifying relationships between various observations or measured parameters and provision of an interim record of field observations to save repeated site inspections (TMR 2012). The above guidelines were followed in this research while sourcing data and information for analyses. A database was created with pavement history, structural and surface condition data, as detailed as possible. For every road, the database included information on traffic category, pavement type, age of pavement, different layers of pavement (including subgrade), thickness of the surface layer, time since last rehabilitation, and method of rehabilitation.

Prozzi and Madanat (2004) suggested that different data sources can be used to develop pavement deterioration models. The major sources are:

- randomly selected in-service pavement sections
- purposely built pavement test sections subjected to the action of actual highway traffic and the environment
- purposely built pavement test sections subjected to the accelerated action of traffic and environmental conditions.

The first two types of data are known as field data, whereas the last is referred to as experimental data. Data from actual in-service pavement sections subjected to the combined actions of highway traffic and environmental conditions are the most representative of the actual deterioration process. All other data sources produce models that are likely to suffer from some kind of bias unless special considerations are taken into account during the estimation of the parameters of the model (Greene 2000).

The most important factors for assessing deterioration of flood-affected roads are historical data for the pre- and post-flood analysis; pavement features such as traffic density, traffic classification, thickness of the surfacing layer and any visible surface condition deterioration. This study considered field data from actual in-service pavement sections subjected to the combined actions of traffic and environmental conditions. The factors that were considered during the sampling and collection of data are as follows:

- rehabilitation works conducted before or after the flooding event
- availability of pre- and post-flood data for same pavement section following the flooding events from 2010 to 2013
- pavement history: age of pavement
- pavement profile: traffic density, thickness of asphalt layer, traffic category
- deflection data from FWD testing
- visible loss of surface condition deterioration - such as rutting, roughness and cracking data or record of such data in the pavement history.

4.4 DATA COLLECTION AND ANALYSIS

This study aimed to gather available FWD deflection and surface condition data of flood affected roads across Australia. FWD deflection data before and after floods were used to evaluate the reduction of the structural strength after flood and rutting and roughness data were used to evaluate post-flood surface condition deterioration of pavements. Two types of data were collected for analyses which were sourced from four organisations:

Observational data: Observational field data were collected from three sources highlighted as follows:

- Brisbane City Council (BCC)
- Department of Transport and Main Roads (TMR), Queensland
- Roads and Maritime Services (RMS), New South Wales,

Brisbane City Council conducted FWD testing before and after flooding on some roads and retested a number of roads regularly after the January 2011 flood. These data were made available for this research. Brisbane City Council has a record of historical data in their PMS which includes deflection data from FWD testing for different chainages and each lane, traffic category, and surfacing layer thickness data. They have provided data for 16 flood affected roads. Among them, only two roads had both pre and post-flood deflection data. Brisbane City Council conducts visual inspections to record surface condition data. As a part of this research, a number of flood affected roads were regularly monitored after the January 2011 flooding event to check the long term impact of flooding and post-flood rehabilitation on pavements. The deflection data was analysed using CIRCLY5.0 (Wardle 2009) to calculate the stiffness moduli of the various pavement layers including the Californian Bearing Ratio (CBR) of the subgrade. With stiffness modulus, the modified structural numbers of pavement sections can then be calculated.

TMR, Queensland provided surface condition data, such as roughness, rutting and cracking for flood affected sections. TMR collects surface condition data (for example roughness, rutting and cracking) at the network level. In recent years, TMR conducted reconstruction works on approximately 8,741 km of the state-controlled road network (TMR 2015). TMR provided a list of road sections where a flood recovery project occurred. This data set includes pavement age and type, pavement width, pavement material type and depth of each layer, and average annual daily traffic (AADT).

RMS NSW provided a list of inundated sites including pre and post-flood FWD data and surface condition data such as roughness, rutting and cracking from 2010 to 2014. Their data included information such as pre-and post-flood deflection data; pavement type; pavement category; surface width; construction year; carriageway type and year of last rehabilitation or resurfacing. They did not have a record of surfacing layer thickness in their PMS. Therefore, Paterson (1987) (see Equation (4.1)) was used to calculate SNC from RMS NSW deflection data.

Published climate data: Climatic condition data such as rainfall data, flood level data Rainfall and temperature data from each site's nearest meteorological station were extracted from the Bureau of Meteorology (BoM 2010a) website.

Pavement subject to flooding is expected to undergo a rapid deterioration phase immediately after a flood. As the study had the historical data, pre- and post-flood data of the road pavement sections, regression based deterministic approach was considered for the model. While assessing pavements for the impact of flooding, a number of strategies were used. First, a number of flood affected roads with before and after flood data or flooded and non-flooded sections were considered for an overall check of the network. Secondly, each road was analysed to determine the statistical significance of the strength or surface condition parameter as applicable to that specific road. Thirdly, highly affected pavement sections were selected for advanced level analysis.

4.4.1 Use of CIRCLY5.0

CIRCLY5.0 was used to calculate layer moduli and subgrade CBR from the surface layer thickness of the pavement and deflection data. The software was used as a tool to calculate deflection values from the surface layer thickness, layer moduli and subgrade CBR. First, surface layer thickness and FWD deflection values for each testing location were recorded from the pavement history file. Layer moduli, CBR values and surface layer thickness were used as the input parameters in CIRCLY5.0. Trial and error method was used to estimate layer moduli and CBR values until the deflection values from CIRCLY5.0 matched the field deflection values. Deflection values were finally obtained as output values from CIRCLY5.0 which were similar to the field deflection values.

Austroads (2010b) recommended Equation (4.1) to compute structural number from FWD Deflection, D_0 for asphalt pavements and Equation (4.2) for pavements with cement treated base:

$$\mathrm{SNC_i} = 3.2 \times \mathrm{D_0}^{-0.63} \quad (4.1)$$

$$\mathrm{SNC_i} = 2.2 \times \mathrm{D_0}^{-0.63} \quad (4.2)$$

where,

$\mathrm{SNC_i}$ = modified structural number at age 'i' (Paterson 1987)

$\mathrm{D_0}$ = maximum deflection (mm) at load centre at age 'i'.

4.4.2 Limitations

There were few limitations in the data analysis which are stated as follows:

- Data received from Brisbane City Council had FWD data. As they conduct visual distress survey, surface condition deterioration data from Brisbane City Council were only used for assessment purposes. These data were not suitable for modelling rutting and roughness.
- RMS NSW were unable to provide any records of layer thickness for flood affected roads therefore CIRCLY5.0 was not used to analyse deflection data. There was a lack of information on RMS NSW data. As a result, these data were limited to general statistical analyses. Section level analyses were not performed on the RMS NSW data.
- The flood affected road database provided by TMR, Queensland had a detailed record of roughness and rutting data before and after flood at the network level. However, no pre- and post-flood deflection data was available for the flood affected pavements. Detailed analyses were conducted based on the TMR surface condition data of flood affected roads. However, due to lack of FWD data, TMR data were not used in modelling deterioration of structural condition.

4.5 STATISTICAL ANALYSIS

The main reason for using various descriptive statistics was to assess and summarise the impacts of floods on roads and compare data with different road characteristics. It also helped in the selection of the specific pavement sections for modelling. SPSS was used for the statistical analyses and comparisons of pre- and post- flood data. Independent samples t-test and paired sample t-test methods were used to compare the structural and surface condition data of flood affected roads. This section will discuss the statistical analysis techniques used in the study.

Mean, standard deviation and coefficient of variation were calculated for maximum deflection and modified structural number of inner wheel path (IWP) and outer wheel path (OWP) of roads using Equations (4.3) and (4.4). The mean is the average number in a total data set and measures the central tendency and can be calculated using Equation (4.3). The standard deviation is the average distance a value is to the mean and can be calculated using Equation (4.4). The coefficient of variation is calculated dividing the standard deviation by the mean (Field 2013).

$$\bar{x} = \sum_{i=1}^{n} \frac{x_i}{n} \tag{4.3}$$

$$S = \sqrt{\sum_{i=1}^{n} \frac{(x_i - \bar{x})^2}{n-1}} \tag{4.4}$$

where,

$\bar{x}$ = Mean

x_i = Sum of sample

n = Number of samples

Correlation coefficient

A correlation coefficient is a descriptive statistic that expresses degree of relationship between two variables. A correlation coefficient provides a quantitative way to express the degree of relationship between two variables. The definition formula is given in Equation (4.5) (Field 2013).

$$r = \frac{\sum z_x z_y}{N} \tag{4.5}$$

Coefficient of determination (R^2)

R square (R^2) value is known as the coefficient of determination. The correlation coefficient is the basis of the coefficient of determination, which indicates the proportion of variance that two variables in a bivariate distribution have in common. The coefficient of determination is calculated by squaring r; it is always a positive value between 0 and 1 (Field 2013).

The closer the fit is to the data points, the closer r-square will be to the value of 1. A larger value of r-square does not necessarily mean a better fit because the degrees of freedom can also affect the value. Thus if more parameters are introduced, the r-square value will rise. However, a higher value of r-square does not necessarily imply a better fitting curve. The adjusted r-square value accounts for the degrees of freedom and is a better measure of the goodness of fit (Field 2013).

Standard Error

The standard deviation of sample means is known as the standard error of the mean (SE) or standard error for short. As such it is a measure of how representative a sample is likely to be of the population. A large error (relative to the sample mean) means there is a lot of variability between the means of different samples and so the sample we have might not be representative of the population. A small standard error indicates that most sample means are similar to the population mean and so our sample is likely to be an accurate reflection of the population (Field 2013).

Confidence Interval

A confidence interval for the mean is a range of scores constructed such that the population mean will fall within this range in 95% of samples. The confidence interval is an interval within which we are 95% confident that the population mean will fall (Field 2013).

4.5.1 Independent and Paired Samples t-Test

Manipulating the independent variable systematically is a powerful research tool because it goes one step beyond merely observing variables. T-test is a method of testing the difference between two means. There are two different t-tests namely independent-samples t-test (or independent t-test) and dependent t-test (called the paired-samples t-test in SPSS Statistics) (Field 2013).

The independent-samples t-test is used when there are two experimental conditions and different participants (or unrelated groups) were assigned to each condition (this is sometimes called the independent–measures or independent-means). The dependent t-test (called the paired-samples t-test in SPSS Statistics) is used when there are two experimental conditions and the same participants or two related groups took part in both conditions of the experiments (Field 2013).

4.5.1.1 Rationale for the t-test

Both types of t-tests have a similar rationale, which is based on hypothesis testing (Field 2013):

- Two samples of data are collected and the sample means are calculated. These means might differ by either a little or a lot.
- If the samples come from the same group, then it is expected that their means to be roughly equal. Although it is possible for their means to differ by chance, large differences between sample means are expected to occur very infrequently. Under the null hypothesis, it is assumed that the experimental manipulation has no effect on the participants: therefore, the sample means are expected to be very similar.
- Standard error is used as a gauge of the variability between sample means. If the standard error is small, then most samples are expected to have very similar means. When the standard error is large, large differences in sample means are more likely. If the difference between the collected samples is larger than expected based on the standard error, then one of two thing can be assumed (Field 2013):
    1. There is no effect and sample means in the group fluctuate a lot and by chance, two samples were collected which were atypical of the population from which they came.
    2. The two samples come from different groups but are typical of their respective parent group. In this scenario, the difference between samples represents a genuine difference between the samples (and so the null hypothesis is unlikely).

Therefore, standard error of the differences between the two means can also be used as an estimate of the error of the model (or the error in the difference between means). Therefore, the t-test statistic is calculated by using Equation (4.6). It can be used to compare the model or effect against the error (Field 2013).

$$t = \frac{(\overline{X}_1 - \overline{X}_2) - (\mu_1 - \mu_2)}{\text{Estimate of the standard error}} \tag{4.6}$$

where,

$\overline{X}_1$	=	Mean of samples from Group 1
$\overline{X}_2$	=	Mean of samples from Group 2
$\mu_1 - \mu_2$	=	expected difference between group means

Differences between the overall mean of the two samples should be examined and compared with expected differences between the means of the two groups from which the samples came. If the null hypothesis is true, then the samples have been drawn from the same group. Therefore under the null hypothesis $\mu_1 = \mu_2$ and therefore $\mu_1 - \mu_2 = 0$ (Field 2013).

In paired samples t-test, the mean difference between samples is calculated ($\overline{D}$) and it is expected to find the difference between group means (μ_D), and then takes into account the standard error of the differences ($s_D / \sqrt{N}$) (refer to Equation (4.7)):

$$t = \frac{\overline{D} - \mu_D}{s_D / \sqrt{N}} \tag{4.7}$$

where,

$\overline{D}$	=	mean difference between samples
μ_D	=	difference between group means
$s_D / \sqrt{N}$	=	standard error of the differences

4.6 CHOOSING THE MODELLING TECHNIQUE

Modelling deterioration of pavements is a complex and tedious task. Flooding condition adds more complexity in the system. As discussed in Chapter two, different types of modelling approach were reviewed for the study. As the pre- and post-flood data were available, the mechanistic-empirical deterministic based approach was chosen for the development of the models in this study. Stepwise linear regression and non-linear regression methods were used for the model development. A brief review of stepwise linear regression and non-linear regression methods are discussed below.

Stepwise linear regression is a method of regressing multiple variables while simultaneously removing those that are not important. Stepwise regression essentially does multiple regression a number of times, each time removing the weakest correlated variable. At the end, the variables that best explain the distribution can be found. The only requirements are that the data are normally distributed (or rather, that the residuals are), and that there is no correlation between the independent variables (known as collinearity) (SPSS Version 22.0 2013).

Non-linear regression is a method of finding a non-linear model of the relationship between the dependent variable and a set of independent variables. Unlike traditional linear regression, which is restricted to estimating linear models, non-linear regression can estimate models with arbitrary relationships between independent and dependent variables. This is accomplished using iterative estimation algorithms. The dependent and independent variables should be quantitative. Categorical variables, such as religion, major, or region of residence, need to be recoded to binary (dummy) variables or other types of contrast variables (SPSS Version 22.0 2013).

4.7 VALIDATION OF REGRESSION MODELS

Model validation is possibly the most important step in the model building sequence. It is also one of the most overlooked. Often the validation of a model seems to consist of nothing more than quoting the R^2 statistic from the fit (which measures the fraction of the total variability in the response that is accounted for by the model). Unfortunately, a high R^2 value does not guarantee that the model fits the data well. Use of a model that does not fit the data well cannot provide good answers to the underlying engineering or scientific questions under investigation (Snee 1977).

Methods to determine the validity of regression models include comparison of model predictions and coefficients with theory, collection of new data to check model predictions, comparison of results with theoretical model calculations, and data splitting or cross-validation in which a portion of the data is used to estimate the model coefficients, and, the remainder of the data is used to measure the prediction accuracy of the model (Snee 1977).

Regression analysis is widely used in data analysis and the development of empirical models. After a regression model which gives an adequate fit to the data has been found, one proceeds to use the model for prediction, or control, or to learn about the mechanism which generated the data. Most will agree, however, that before a model is used, some checks of its validity should be made. Snee (1977) mentioned that

"a review of the statistical literature to see what kinds of model checks are recommended offers little guidance except the admonishment that model validation is a good thing to do. In many instances analyses reported do not mention how or if the model was validated".

Data splitting, or cross-validation, is an effective method of evaluating a regression model. All available model validation procedures should be used when it is practical to do so. The following procedures have been found useful in checking the validity of a regression model (Snee 1977).

1) Comparison of the model predictions (y) and coefficients (a) with physical theory
2) Collection of new data to check model predictions
3) Comparison of results with theoretical models and simulated data
4) Reservation of a portion of the available data to obtain an independent measure of the model prediction accuracy.

4.7.1 Check on Model Predictions and Coefficients

A check on the model predictions and coefficients should be made as soon as the model has been developed. Negative predictions of a theoretically positive quantity or coefficients with wrong signs are indications of an inappropriate or poorly estimated model. In some instances it may not be necessary to predict outside the region of the data; however, in other situations, predictions outside the region of the available data are of great interest (Snee 1977).

4.7.2 Collection of New Data

Another good method of model validation is the collection of new data which can be compared with the predictions of the model. The validity of the mathematical and physical assumptions used in developing a model and estimating the coefficients is less open to question if a model gives accurate predictions of new data. In effect, the collection of new data provides an overall check on the entire model construction process (Snee 1977).

4.7.3 Comparison with Theory and Simulation Results

In some instances, theory may exist which will enable us to obtain insight as to whether the model makes sense. The use of theory will most often give information as to direction and relative magnitude of effects. In some instances, a theoretical model may exist but may be too complicated for practical use, or it may ignore complexities present in the experimental situation. When available, theory should be used to check the accuracy of an empirical model (Snee 1977).

4.7.4 Data Splitting or Cross-validation

Most will agree that of the methods discussed above, the collection of new data is the preferred method for model validation. In many instances this is neither practical nor possible. In this situation a procedure which simulates the collection of new data is needed. A reasonable way to proceed is to split the data in hand into two sets. The first set of data, called the estimation data, is used to estimate the model coefficients. The remaining data points, called the prediction data, are used to measure the prediction accuracy of the model (Snee 1977). A half-and-half split appears to be the most popular method. Data splitting provides a data set to measure the in-use prediction accuracy of the model and simulates the complete or partial replication of a study.

Two methods from the above mentioned model validation techniques were used to check the models developed in this study: a comparison of the model predictions (y) and coefficients (a) with physical theory, and data splitting or cross-validation.

4.8 SUMMARY

This chapter presented the methodology followed in this study to assess the flood affected pavements and then develop the models using the existing data. The study used in-field data which were collected following the recent flooding events. There were some limitations to the data which are discussed. Lack of pre- and post-flood data for moisture condition of the flood affected pavements made it impossible to include the moisture content parameter into the models. However, it is indeed a useful parameter to find out when the pavement is in its weakest condition following a flooding event.

5. ASSESSMENT AND MODELLING DETERIORATION OF STRUCTURAL CONDITION OF FLOOD AFFECTED PAVEMENTS

5.1 ASSESSMENT OF STRUCTURAL CONDITION OF FLOOD AFFECTED PAVEMENTS

This chapter enhances the understanding of the impacts of floods on pavements by assessing the structural condition data of flood affected pavements of Brisbane City Council and Roads and Maritime Services of New South Wales (RMS NSW). A review of the available literature on the flood affected road network of Brisbane City Council impacted by the January 2011 flood was included in Section 3.3. As a part of this research project, a number of flood-affected roads were regularly tested by Brisbane City Council following the January 2011 flood. There was a significant difference between the FWD data values for each lane of the same pavement section before and after flooding. The distinction between flooded points and non-flooded points for each pavement type was based on the flooding conditions prevalent on January 13, when flooding was at its greatest extent. Two models were then developed to predict the rapid deterioration of structural condition of flood affected pavements.

The January 2011 flooding was a river flood; water rose gradually and relatively slowly. River water deposited mud on the road and slowly washed the area. There was no prolific scouring often associated with rapid rise of river water or flash flooding. The damage was more likely to be due to loading / trafficking of the saturated pavement and moisture entering the pavement.

Brisbane City Council conducted the FWD testing on several flood affected pavements after the January 2011 flood. FWD testing was undertaken in all wheel paths to determine the effect of flooding on the pavement. Six roads in the Brisbane City Council area were included in this analysis: Luxford Street (120 m) and Sutton Street (133 m), Chelmer; Aldersgate Street, Oxley (370 m); Haig Road, Milton (275 m) and Auchenflower (370 m); and Cordelia Street in South Brisbane (885 m). These roads were selected based on the availability of the pre- and post-flood data, pavement type, traffic category and extent of inundation.

Table 5.1 presents the pavement history of these streets including pavement age and type, asphalt layer thickness, the extent of inundation, year and total length tested for FWD, number of points tested and date of rehabilitation. The traffic density categories of these roads range from cul-de-sac, residential collector, industrial access road to arterial road.

The typical layout of strength testing of Brisbane City Council streets is shown in Figure 5.1. In the analysis and discussion of FWD data of any wheel path of Brisbane City Council streets, 1L and IR represent the outer wheel path (OWP) and inner wheel path (IWP) of lane 1, respectively, and 2L and 2R represent the inner wheel path and outer wheel path of lane 2, respectively.

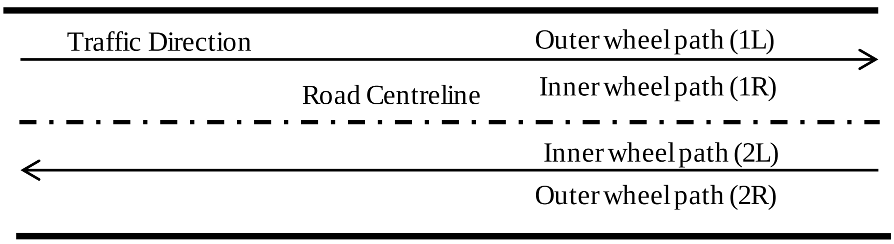


Figure 5.1: Typical layout of strength testing of BCC streets

All FWD deflection data were corrected for temperature (as applicable) following Figure 2.4 in Section 2.3.2 (TMR 2012). All FWD data tested with 50-KN loading were converted to 40-KN loading before the comparison.

5.2 SUMMARY OF RAINFALL DATA

The monthly rainfall is the total of all available daily rainfall for the month. Observations of daily rainfall are nominally made at 9 am local clock time and record the total for the previous 24 hours. Rainfall includes all forms of precipitation that reach the ground, such as rain, drizzle, hail and snow. Rainfall data for Brisbane from 2009-2015 is shown in Table 5.2 (BoM 2016a) and confirms that December 2010 had the highest amount of rainfall in the previous 5 years.

Table 5.1: Pavement history of six Brisbane City Council streets

Road Name	**Year of construction**	**Pavement Type**	**Subgrade Type**	**Traffic Category**	**ESA**	**AC layer Thickness (mm)**	**Extent of Inundation**	**Length tested (m)**	**No of Chainages tested**	**FWD Data Collection**	**Latest Rehabilitation**
Luxford Street, Chelmer	1975	Thin AC & gravel	Clay (with/without sand)	Residential Collector	$3.7x10^4$	40-65	Total	133	59	2010 2011 2014	26/5/2011
Aldersgate Street, Oxley	1973		Silt with sand/ clayey silt			20-40	Partial	380	31	2011 2013 2014	2011
Sutton Street, Chelmer	1975		Clay (with/without sand)	Cul-de-Sac/dead end	$1.5x10^4$	40-65	Partial	133	76	2011	2011
Haig Road, Milton	1989	AC & 200CTB	Clayey Gravel with sand	Industrial Access	$1.5x10^6$	15-75, (Ave. 40 mm)	Total	275	52	2010 2011 2013 2014	2012
Haig Road, Auchenflower	1988	Thin AC & gravel	Clayey Gravel			50-60	Partial	370	60	2011 2013 2014	14/8/2012
Cordelia Street, South Brisbane	2010 & 2002	190AC& gravel	Clayey Gravel with sand/Gravelly clay with sand	Arterial	$3.7x10^6$	190, 180 & 60-130	Partial	885	124	2011 2013 2014	11/09/2011, 02/09/2012, 15/10/2014, 31/05/2015
Total Length and chainages tested=								2176 m	402		

Table 5.2: Average Monthly Rainfall (mm) for Brisbane area (BoM 2016a)

Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
2009	77.4	131.8	47.6	195.2	240.8	87.6	3.6	4.0	23.4	57.4	31.4	172.2	1072.4
2010	40.2	272.4	162.2	36.4	61.2	6.2	27.8	104.6	103.6	306.4	57.8	479.8	1658.6
2011	315.8	157.0	149.2	100.2	69.2	10.6	12.4	76.8	18.6	117.8	15.6	132.4	1175.6
2012	343.4	162.2	118.2	149.0	36.4	122.0	51.2	0.2	5.6	21.4	116.8	50.8	1177.2
2013	281.6	250.6	172.4	104.6	44.0	59.4	39.0	0.2	13.2	22.6	104.8	19.2	1111.6
2014	143.8	15.8	164.4	14.2	28.6	16.2	14.2	93.2	27.6	5.8	142.0	124.4	790.2
2015	153.9	142.5	109.2	70.8	70.4	56.1	23.7	41.2	30.4	71.4	104.6	132.7	1006.3
Station: Brisbane, Number: 40913, Opened: 1999, Lat: 27.48° S, Lon: 153.04° E, Elevation: 8 m													

5.3 LUXFORD STREET IN CHELMER

Brisbane City Council conducted FWD testing on Luxford Street in Chelmer (Rosebery Terrace to Queenscroft Street) to investigate and determine the impact of the January 2011 flood on the condition and life of the pavement. The street was totally flooded during the January 2011 flooding, was included in the FWD testing program and subsequently in 2010/2011 budget for rehabilitation. Pavement history records indicated that the street was last resurfaced with asphalt in 1975. Local Asset Services had previously carried out maintenance in the form of AC patching.

The existing surface is believed to be 41-year old asphalt. The section between chainage 16 and chainage 63 had extensive patching with the patches over a stormwater pipe having severe depressions. A report from a local resident indicates that depressions were present in this street but they became more severe after flooding. Recent patching continues to settle. Subgrade is classified as Clay (with/without sand) in all sections and moisture contents above the plastic limit make the pavement sensitive to moisture. The street is a residential collector (ESA 3.7×10^4) and is relatively lightly trafficked. Pit sampling and asphalt coring (undertaken in December 2010) indicate composition of the two sections as shown in Table 5.3.

Table 5.3: Thickness of surface layer (Luxford Street, Chelmer)

Section 1 (Ch. 0-65 m)	**Section 2 (Ch. 65-133 m)**
45-65 mm AC in Original Pavement (65-145 mm in Patched area)	40-60 mm AC in Original Pavement
135-150 mm Gravel with Sand & Silt	190-200 mm Gravel with Sand & Clay
170 mm Clayey Gravel with Sand	
370-400 mm (Total thickness)	240-250 mm (Total thickness)

The street was highly affected during the January 2011 flood. A visual inspection of surface condition was completed on the street six weeks after the flood. Some areas of crocodile cracking without depression and extensive depressions over the storm water pipe were observed. Another inspection of the street in March 2011 indicated a small area of additional crocodile cracking which developed after the flood. The pavement condition data of this street is summarised in Table 5.4. A small increase (0.4%) in pavement failures was observed following the flood.

Table 5.4: Pavement condition data of Luxford Street, Chelmer

Inspection Date	Pavement Failures	Surface Failures
7/12/2010	1.9%	2.1%
23/02/2011	2.3%	1.3%

5.3.1 Analysis of Deflection Data

Sample size, mean and standard deviation of the deflection and modified structural number was calculated for Luxford Street (Table 5.5). If flooding had any effect on pavement, the affected sections should have a higher deflection, a lower modified structural number (SNC), and lower subgrade resilient modulus than prior to flooding. CIRCLY5.0 was used to calculate layer moduli and subgrade CBR from surface layer thickness and deflection. SNC was calculated using Equation (4.1) as given in Paterson (1987).

The flood reached its peak in Brisbane on 13 January, 2011. Most of the rainfall fell in 5 days from 9 to 13 January and flood water started to recede from 14 January. Roads were under the effect of heavy rainfall and flooding for six days. FWD deflection testing of Luxford Street was undertaken on 24 February 2011. The FWD maximum deflection data versus chainage and structural number versus chainage graphs of two lanes of Luxford Street are presented in Figures 5.2 to 5.5. These graphical representations clearly illustrate that deflection values were higher immediately after flooding and there were decreases in the SNC values after flooding. These graphs indicate that structural strength of the pavement section reduced significantly after flood. The reduction of structural numbers in different pavement sections ranged from 1.5% to 50%. This indicates the need for further investigation of the data to quantify the reductions, such as analysing deflection data at chainages where major changes in deflection and SNC values were observed after the flood.

Table 5.5: Mean and standard deviation of Maximum Deflection (D_0, mm) and SNC of Luxford Street, Chelmer

Name of Road, Wheel Path and Lane	Sample Size (N)	Parameter	D_0 (mm)				SNC			
			Nov, 2010	24 Feb, 2011	9 Dec, 2014	23 Mar, 2016	Nov, 2010	24 Feb, 2011	9 Dec 2014	23 Mar, 2016
Luxford Street, Chelmer	59	Mean	1.057	1.364	0.871	0.928	3.43	2.96	3.95	3.75
		STD	0.486	0.698	0.430	0.426	1.00	0.913	1.553	1.136
1L-Outer WP	15	Mean	1.301	1.914	0.823	0.884	2.954	2.539	3.885	3.840
		STD	0.570	1.029	0.340	0.408	0.783	1.018	0.898	1.161
1R-Inner WP	14	Mean	0.977	1.238	0.833	0.945	3.688	3.105	3.981	3.683
		STD	0.463	0.505	0.383	0.429	1.296	1.029	1.200	1.120
2L-Inner WP	15	Mean	0.937	0.921	0.894	0.953	3.611	3.627	3.943	3.790
		STD	0.437	0.371	0.423	0.493	0.848	0.896	1.499	1.315
2R-Outer WP	15	Mean	1.008	1.146	0.806	0.930	3.479	3.107	4.078	3.697
		STD	0.422	0.358	0.384	0.425	0.969	0.741	1.216	1.083
*Sample sizes refer to total number of chainages and STD refers to Standard Deviation										

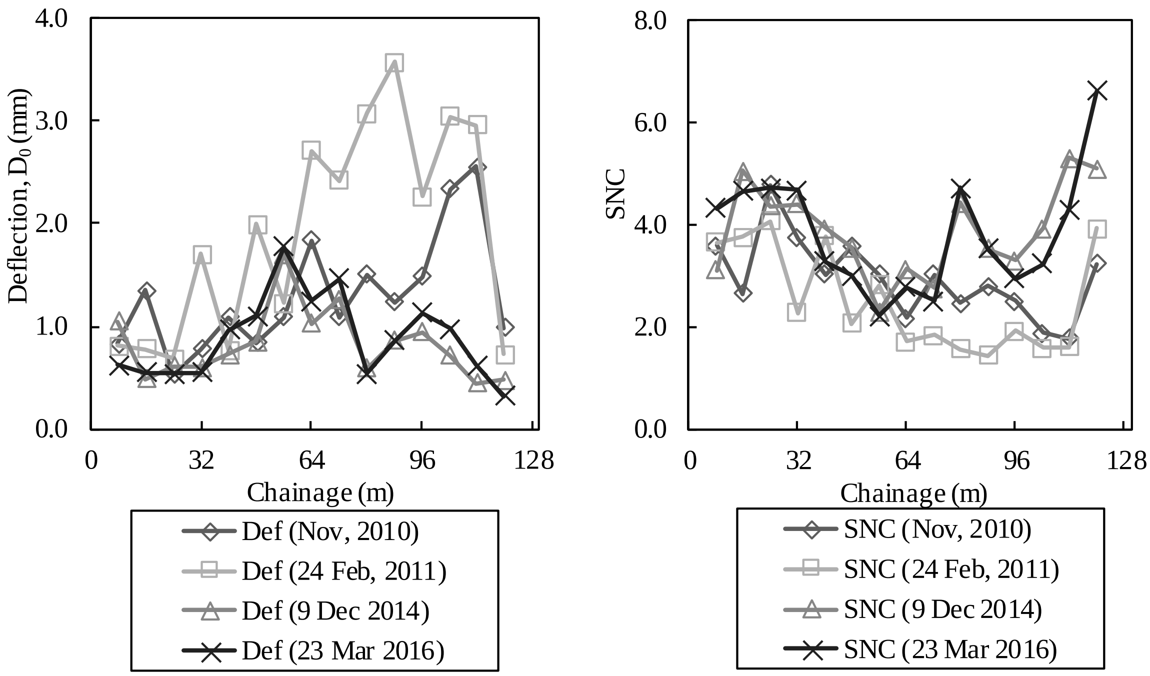


Figure 5.2: Maximum deflection vs chainage and modified structural number vs chainage of outer wheel path (1L) chainage (Luxford Street, Chelmer)

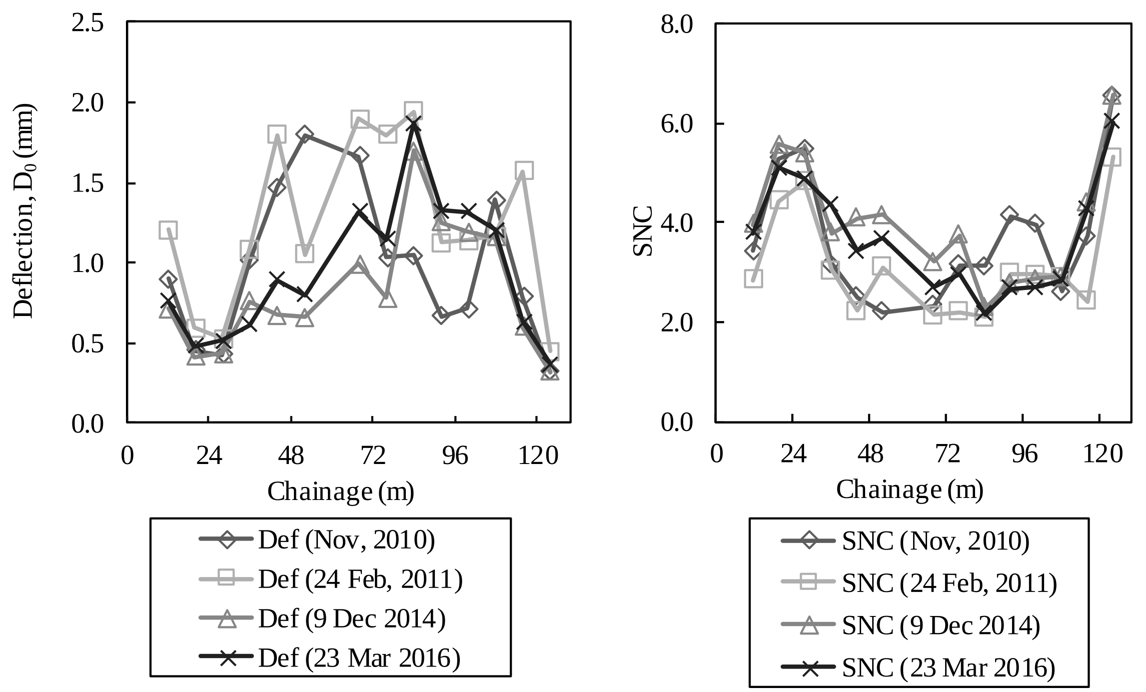


Figure 5.3: Maximum deflection vs chainage and modified structural number vs chainage of inner wheel path (1R) (Luxford Street, Chelmer)

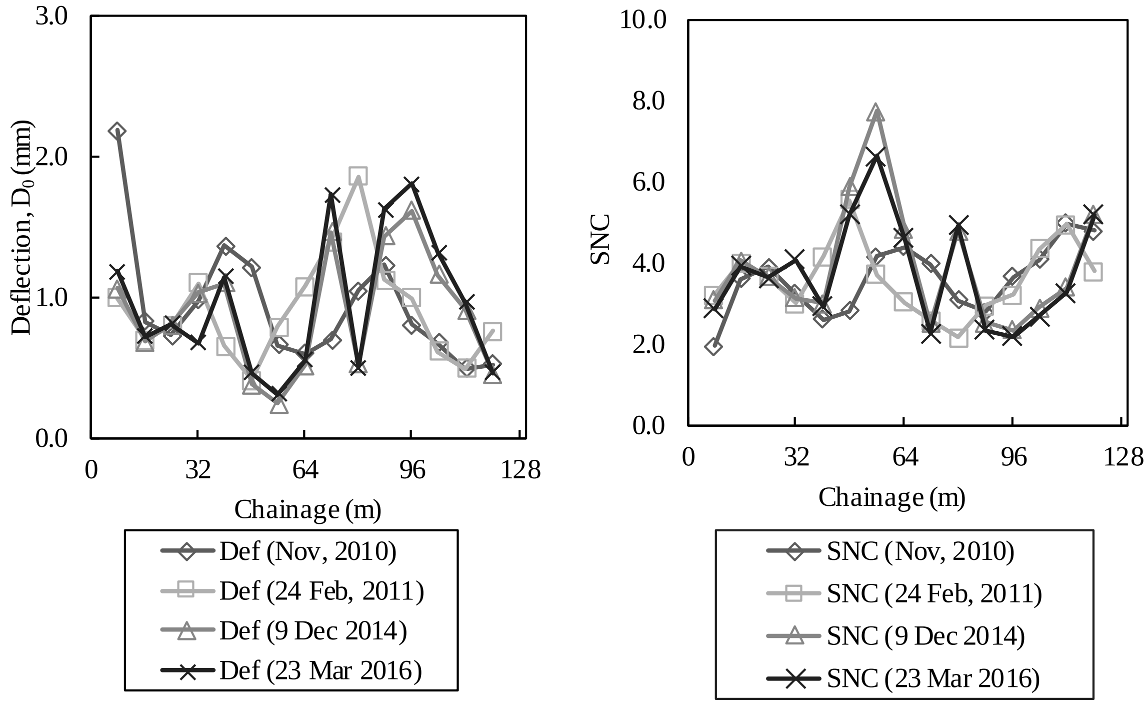


Figure 5.4: Maximum deflection vs chainage and modified structural number vs chainage of inner wheel path (2L) (Luxford, Street, Chelmer)

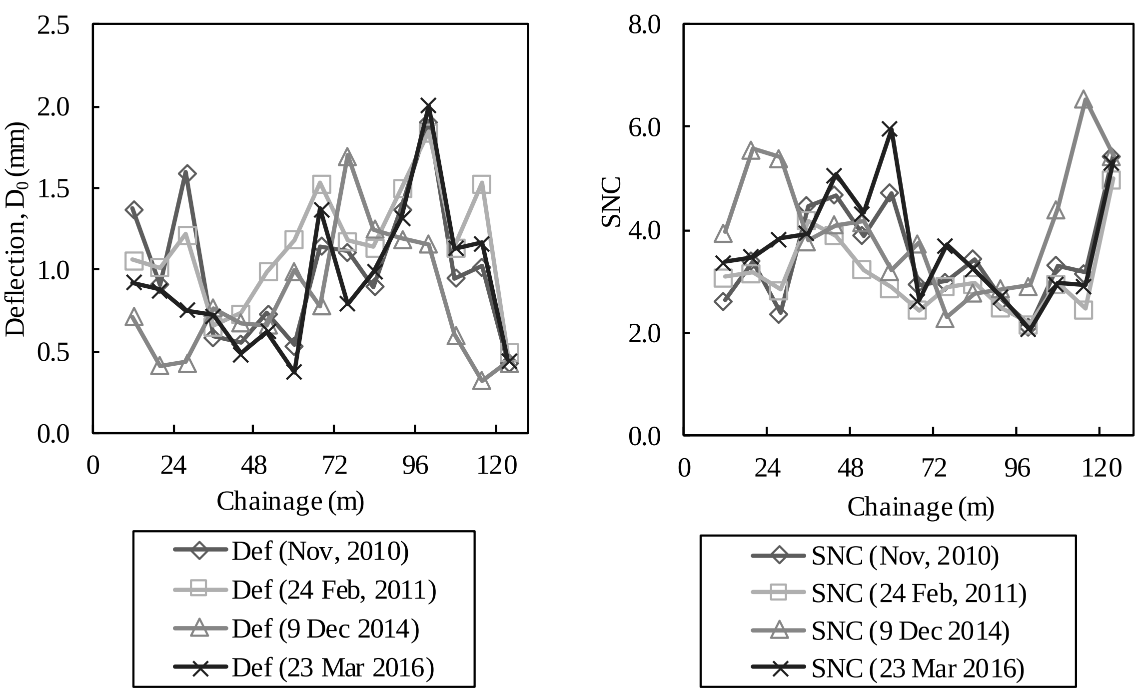


Figure 5.5: Maximum deflection vs chainage and modified structural number vs chainage of outer wheel path (2R) (Luxford Street, Chelmer)

The street was rehabilitated on 26 May 2011. The percentage increase or decrease in strength immediately after flooding, three years post-flooding and five years post-flooding are shown in Figures 5.6 to 5.9. Positive signs refer to an increase and negative signs refer to a decrease in strength. These figures indicate that the pavement regained strength from 10% to 100%.

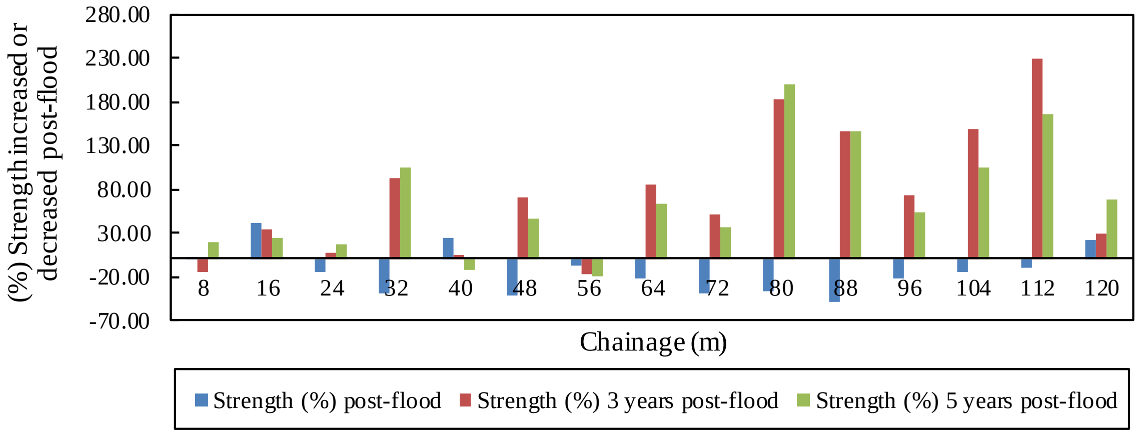


Figure 5.6: Comparison of structural strength post-flooding (outer wheel path [1L]) in Luxford Street, Chelmer

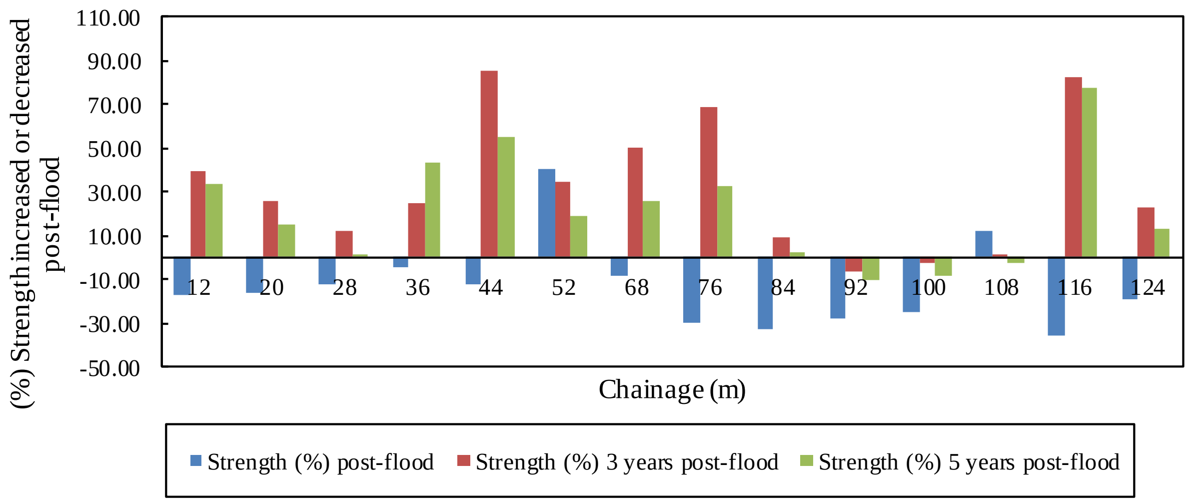


Figure 5.7: Comparison of structural strength post-flooding (inner wheel path [1R]) in Luxford Street, Chelmer

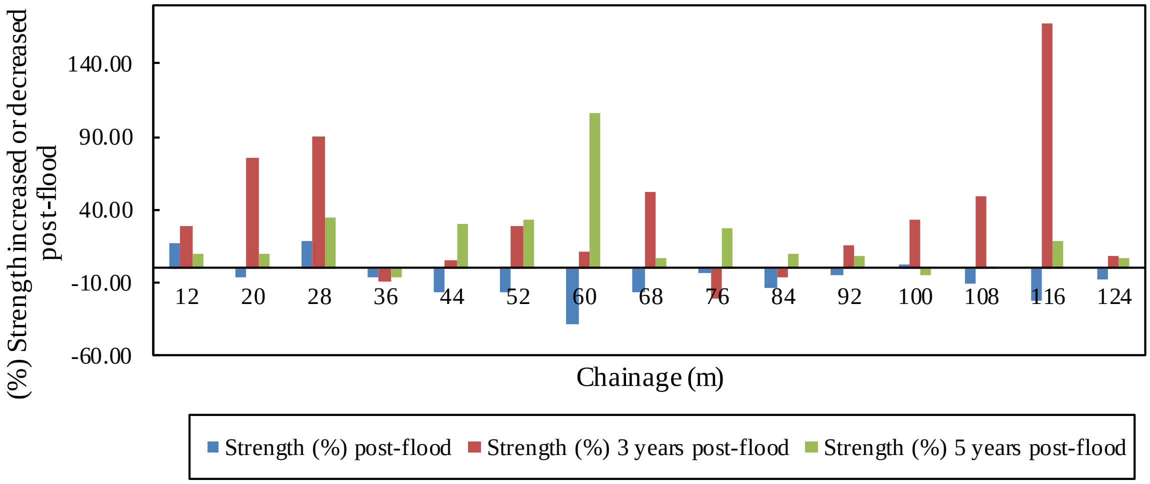


Figure 5.8: Comparison of structural strength post-flooding (outer wheel path [2R]) in Luxford Street, Chelmer

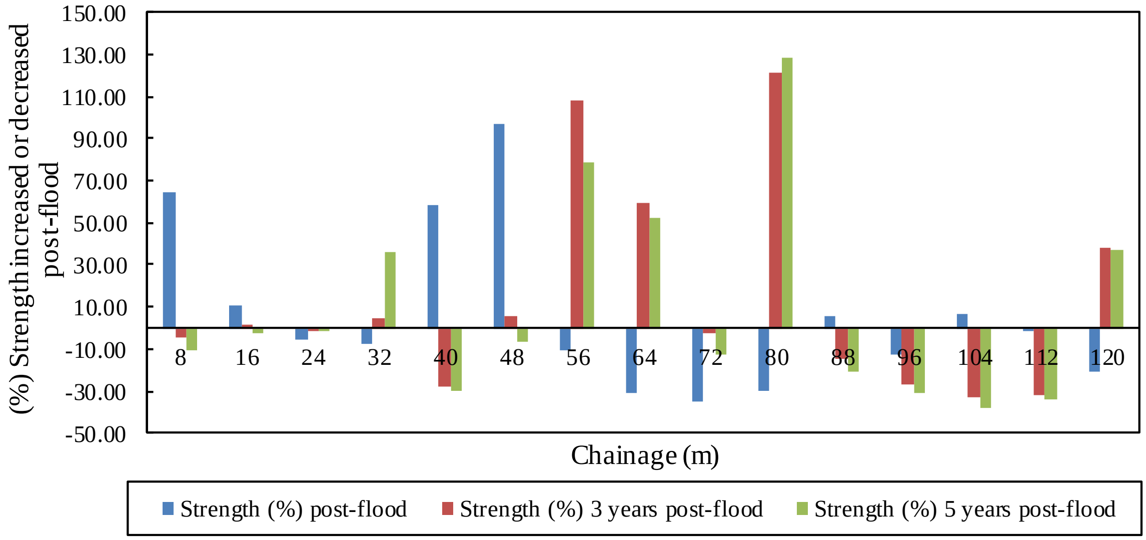


Figure 5.9: Comparison of structural strength post-flooding (inner wheel path [2L]) in Luxford Street, Chelmer

5.3.2 Loss of Subgrade Strength

The inundation during flooding resulted in significant losses in subgrade strength. The deterioration curves for losses of subgrade strength in Luxford Street are shown in Figures 5.10 and 5.11. There were reductions in subgrade strength from 7% to 68% at different chainages. At some chainages, there were big decreases in CBR (California Bearing Ratio) values.

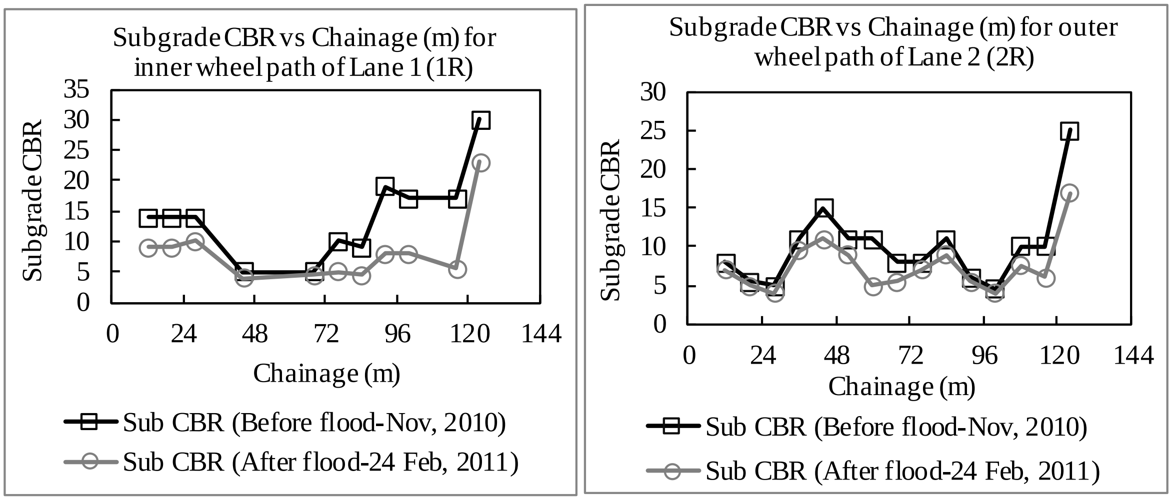


Figure 5.10: Deterioration curves for subgrade response in Luxford Street, Chelmer

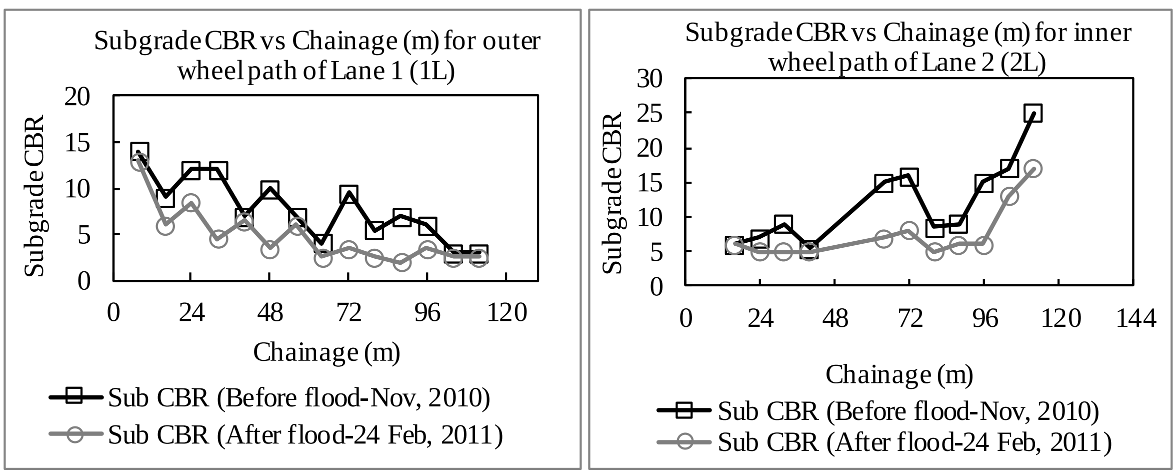


Figure 5.11: Deterioration curves for subgrade response in Luxford Street, Chelmer (inner and outer wheelpath of Lane 1 and 2)

It should be noted that not all sections of this road were highly deteriorated. Some highly deteriorated sections were identified and analysed. The layer Moduli and CBR of some of these pavement sections were calculated using CIRCLY5.0 and are shown in Table 5.6. In Table 5.6, the first column refers to the ID of the pavement section, the first part of the ID refers to the wheel path and second part refers to the chainage (m). The analysis of layer Moduli and CBR of these pavement sections were cross-checked by using EFROMD2 (Vuong 1992). Values obtained from CIRCLY5.0 and EFROMD2 were very similar. There were significant reductions in subgrade strength immediately after the flooding. The calculation of the layer modulus from CIRCLY5.0 indicates a decrease in strength in the Asphalt and Granular layer in some sections. The sections were rehabilitated in 2011 to restore the subgrade strength. The deflection data indicated improvement in strength as a result of rehabilitation in 2011 and dry weather period in the area after the January 2011 flood.

In summary, analyses indicate that many chainages along Luxford Street lost a significant amount of their subgrade strength within six weeks. Some of these chainages were reanalysed using CIRCLY5.0 to assess the trend in the change of deflection and structural number (Figure 5.11) over time. Figure 5.12 is a graphical representation of maximum deflection, SNC and subgrade CBR curves against time for chainage 12, inner wheel path of lane 1 of Luxford Street, Chelmer.

Table 5.6: Calculation of Layer Moduli and CBR Using CIRCLY5.0 for Luxford Street, Chelmer

ID	Date	Thickness of AC Layer (mm)	Thickness of Granular layer (mm)	Asphalt Modulus (MPa)	Granular layer Modulus (MPa)	Subgrade CBR (%)	D_0 (mm)	SNC
1R-12	8/12/2010	50	135	1500	150	14	0.89	3.44
	24/02/2011	50	135	1500	150	9	1.184	2.88
	9/12/2014	65	135	2000	200	14	0.721	3.93
1R-20	8/12/2010	140	145	2000	200	14	0.456	5.25
	24/02/2011	140	145	2000	200	9	0.599	4.42
	9/12/2014	140	145	2500	200	14	0.428	5.46
1R-28	8/12/2010	140	145	2500	200	14	0.428	5.46
	24/02/2010	140	145	2500	200	14	0.428	5.46
	9/12/2014	140	145	2300	200	10	0.522	4.82
2R-60	8/12/2010	140	260	2000	250	9	0.546	4.69
	24/02/2011	140	260	1800	250	3	1.189	2.87
	9/12/2014	140	260	2000	250	3.5	1.074	3.06
2L-64	8/12/2010	130	260	2000	200	8.5	0.606	4.39
	24/02/2011	130	260	1500	200	4	1.047	3.11
	9/12/2014	140	260	2500	200	9	0.534	4.75

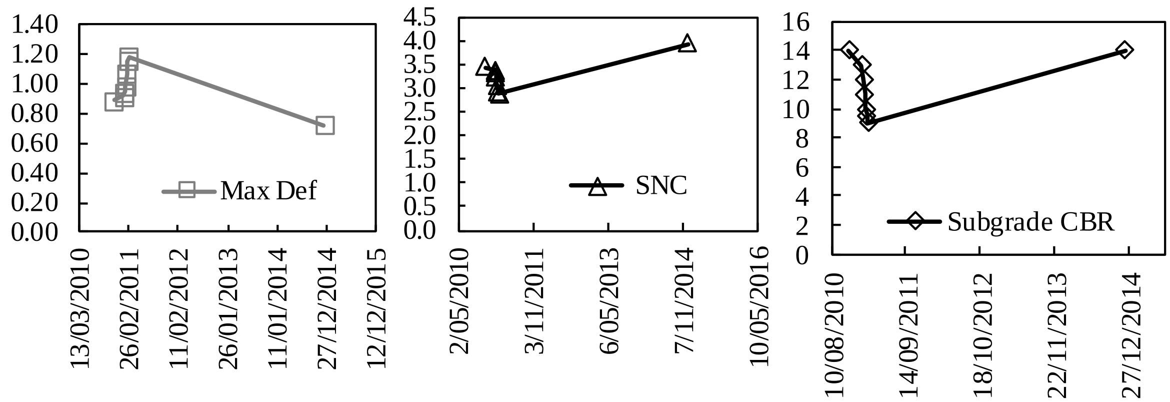


Figure 5.12: Maximum deflection, SNC and Subgrade CBR curve over time for Luxford Street, Chelmer, inner wheel path 1R, Chainage 12

Figure 5.12 shows that deflection values significantly increased and SNC greatly decreased within a relatively short period (six weeks). Subgrade CBR decreased from 14 to 9, the maximum deflection value increased from 0.89 mm to 1.184 mm (33.03% increase) and SNC decreased from 3.44 to 2.88 (16.3% decrease) in six weeks.

Deflection versus time and SNC versus time graphs are shown in Figures 5.13 to 5.15. There were significant increases in deflection value and decreases in SNC values within six weeks of the flood. These deflection values and reductions in SNC values would not have been so high over such a short period of time if the road were to deteriorate under normal weather conditions without the flooding event. The decrease in pavement strength after the flood could lead to a reduction in the service life of the pavement sections if no rehabilitation measures or repairs were undertaken.

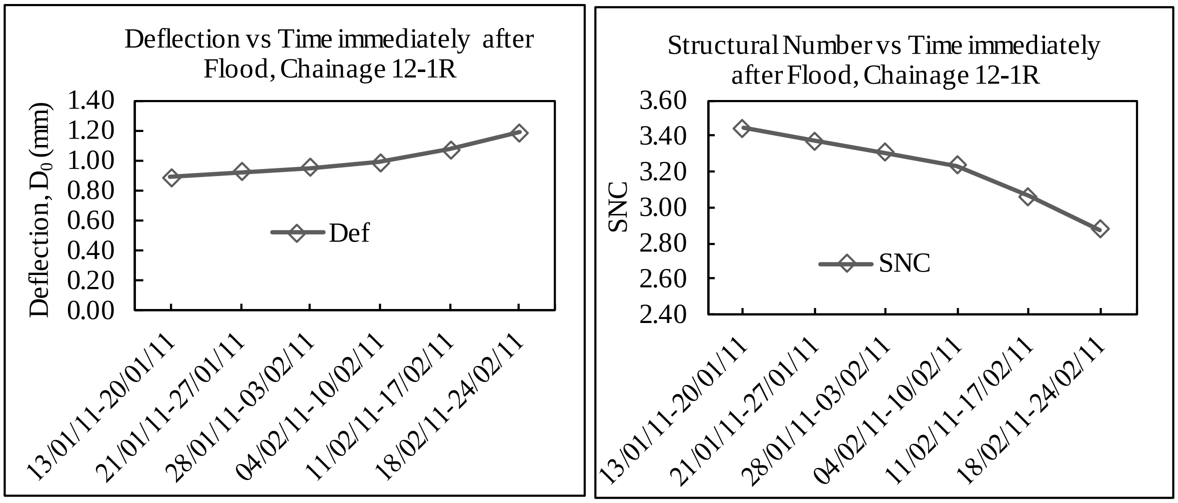


Figure 5.13: Deflection vs time and SNC vs time for Luxford Street, Chelmer, Chainage 12

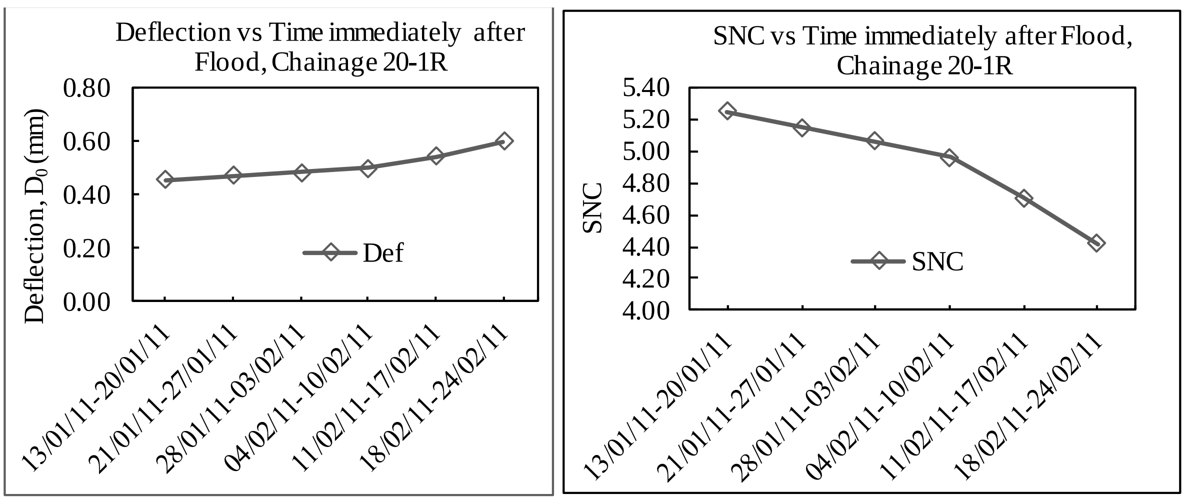
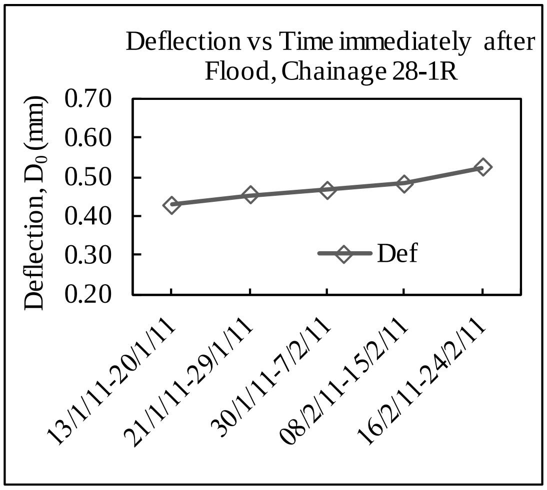


Figure 5.14: Deflection vs time and SNC vs time for Luxford Street, Chelmer, Chainage 20



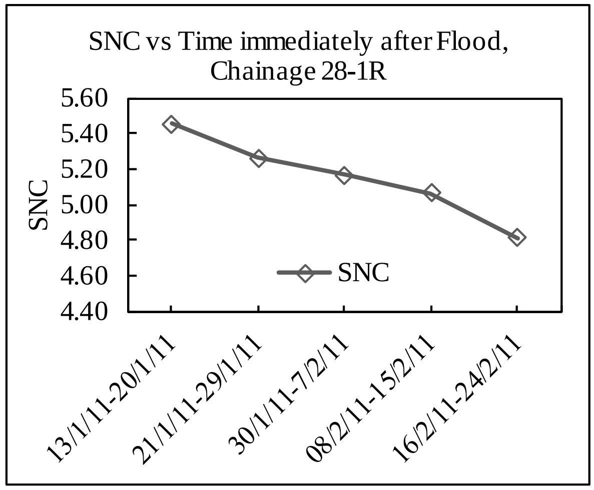


Figure 5.15: Deflection vs time and SNC vs time for Luxford Street, Chelmer, Chainage 28

5.3.3 Summary

Luxford Street in Chelmer, Brisbane was highly affected by flood due to reduction in structural and subgrade strength. Results of the FWD testing showed that inundation reduced the strength of the pavement. The flood-affected pavement sections had a rapid reduction in structural and subgrade strength. The results indicate that the reduction in strength occurred mainly in the subgrade. Maintenance and repair of the pavement is necessary to preserve the integrity of the pavement. Decreases in the quality and serviceability of these pavements would be more rapid than their previous normal rate if left untreated.

5.4 HAIG ROAD IN MILTON

Brisbane City Council conducted an investigation to determine the impact of the January 2011 flood on the condition and life of the pavement of Haig Road in Milton (Baroona Street to Bayswater Street) after Haig Road (Milton) was totally inundated during the flood. This road was included in the FWD testing program and FWD testing was undertaken on 19 December 2010. This road is an industrial access road with ESA 1.5×10^6. It is a directional road with more westbound than eastbound traffic. The existing surface is believed to be 25-year old asphalt (since 1989) in non-reconstructed areas. The RSI traffic category of “E” provides a suitable representation of the current traffic volume for this section.

Pit sampling (boreholes) and asphalt coring undertaken on 19 November 1986 indicated a minimum 200 mm of pavement including 15 mm to 70 mm of asphalt. The thickness of surface layer is shown in Table 5.7. This street has a cement treated base (CTB). CBR of the subgrade is estimated as 12%.

Table 5.7: Thickness of surface layer of Haig Road, Milton

AC	15-75 mm (Ave. 40 mm), 70, 50, 15, 35, 30 mm
Gravel	Variable (Ave. 160 mm)
Total thickness	200 mm

5.4.1 Analysis of Deflection Data

FWD testing was undertaken at five different times; 19 December 2010, 3 April 2011, 26 March 2013, 8 December 2014 and 22 March 2016. Graphical representation of the maximum deflection (D_0) versus chainages and SNC versus chainages are shown in Figures 5.16 to 5.19. There were increases in deflection and decreases in SNC after flooding in many chainages. The first set of testing was done while there was heavy rainfall in the area. However, post-flood FWD testing was done three months after the flooding; as a result, the loss of structural strength is not as high as for Luxford Street, Chelmer. This indicates that pavement regains some strength when it dries up. This road also has CTB which may also have contributed to better pavement performance against flooding in comparison to Luxford Street. At the time of the third testing, performed two years after flooding, some pavement sections had regained more than 50% strength.

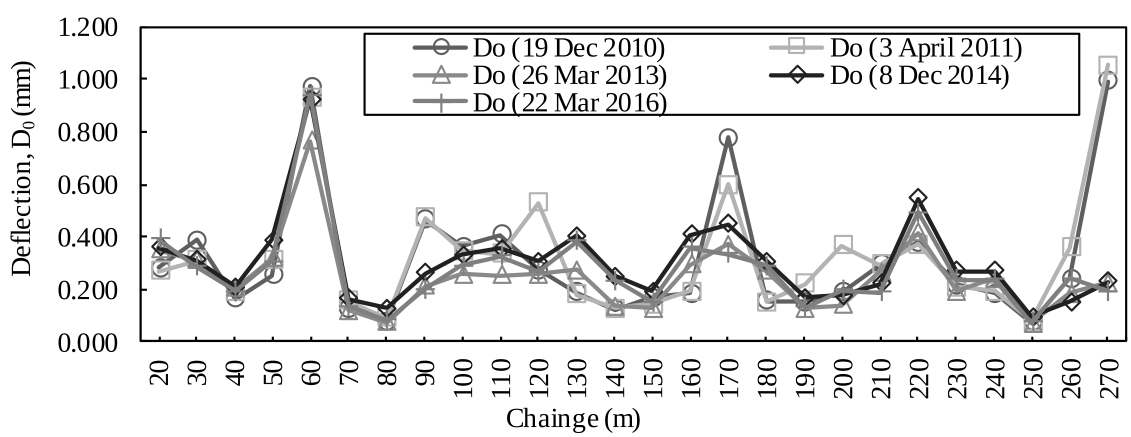


Figure 5.16: Maximum deflection vs chainage (outer wheel path [1L]) of Haig Road, Milton

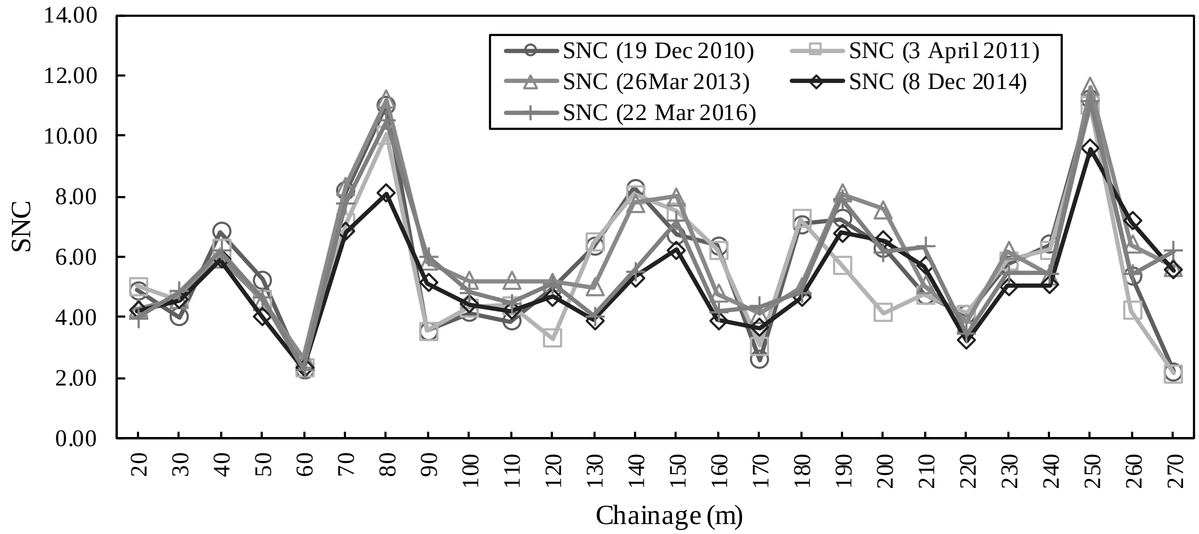


Figure 5.17: Modified structural number vs chainage (outer wheel path [1L]) of Haig Road, Milton

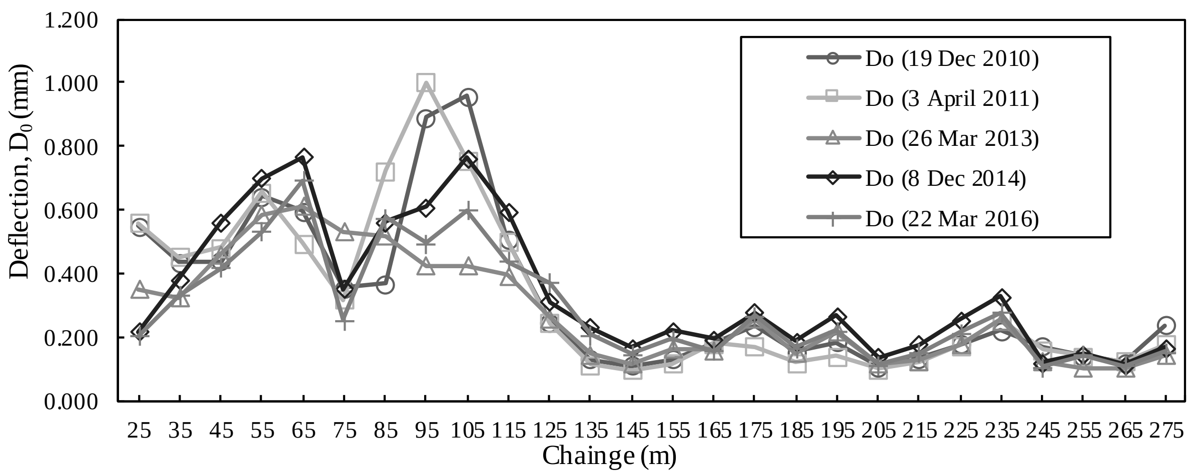


Figure 5.18: Maximum deflection vs chainage (inner wheel path [1R]) of Haig Road, Milton

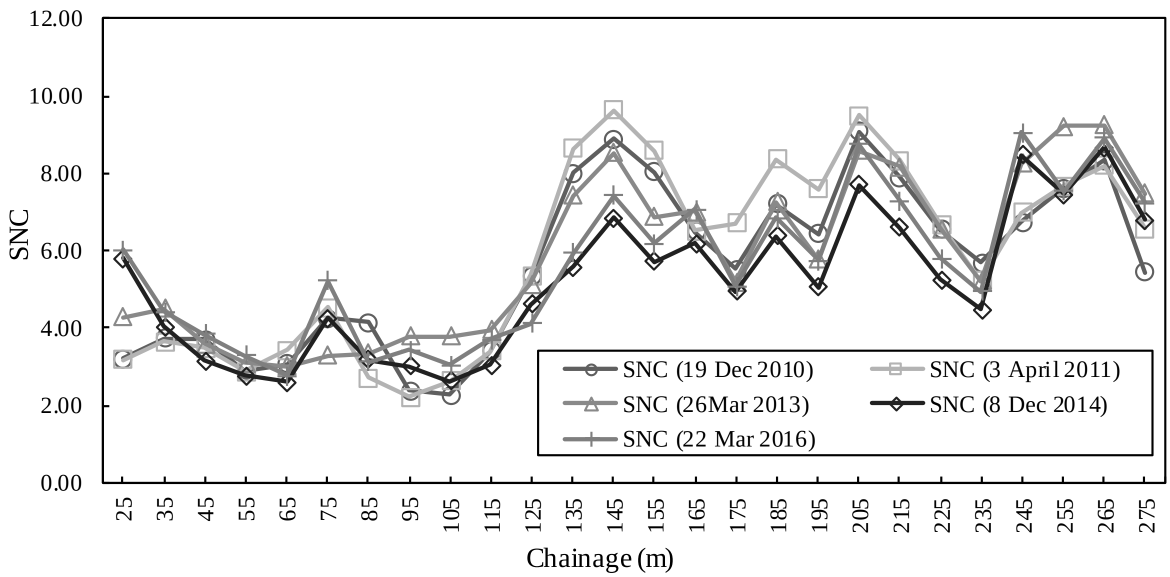


Figure 5.19: Modified structural number vs chainage (inner wheel path [1R]) of Haig Road, Milton

The pavement history file shows that a proposal was made to rehabilitate the road after the flood. The proposed rehabilitation method was cold plane 50 mm, selected pavement failure repairs with 100 mm Type 4 Multigrade AC, spray seal and then 50 mm Type 3 multigrade AC surfacing. However, the resurfacing date of the pavement section was 2012 indicating that it was resurfaced after 24 February 2011. Therefore, it can be concluded that resurfacing also contributed to the strength gain of the pavement. The improvement in pavement strength is clearly visible from the 2013 FWD testing data.

The changes in strength immediately after the flood, 2 years post-flood, 3 years post-flood and 5 years post-flood are shown in Figures 5.20 and 5.21. Positive signs refer to an increase and negative signs refer to a decrease in strength. Of the 26 outer wheel path testing points, 12 testing points gained 3% to 20% strength 3 months post-flood. Seventeen testing points significantly gained 1% to 165% strength 2 years post-flood, only ten testing points gained strength 3 years post-flood and fifteen testing points gained strength 5 years post-flood. Among 26 inner wheel path testing points, eighteen testing points gained 0.25% to 22% strength 3 months post-flood. Thirteen testing points gained 2.8% to 72% strength 2 years post-flood, only seven testing points gained strength 3 years post-flood and some of the sections started to decrease in strength again. Thirteen testing points gained strength 5 years after flood.

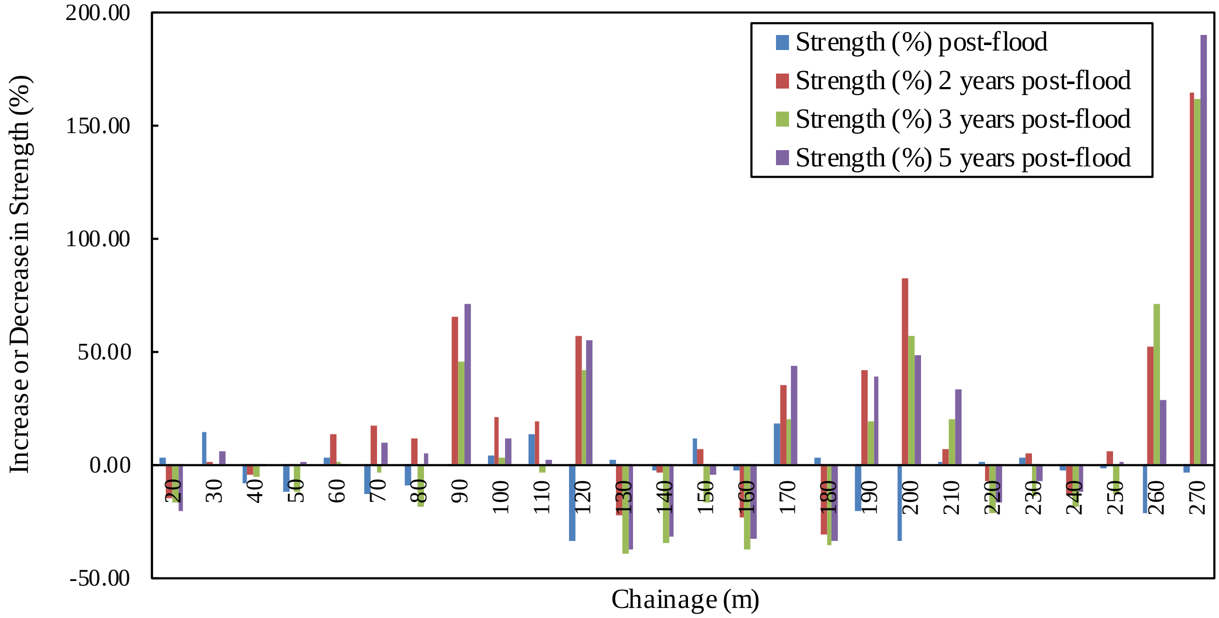


Figure 5.20: Comparison of structural strength post flood (outer wheel path [1L]) in Haig road, Milton

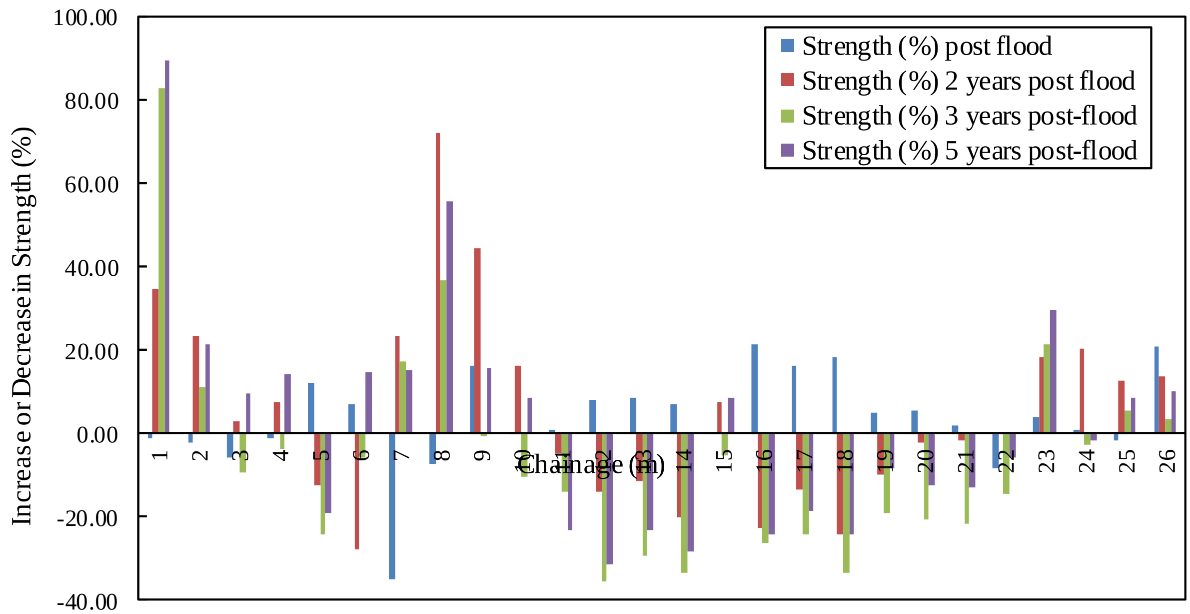


Figure 5.21: Comparison of structural strength post flood (inner wheel path [1R]) in Haig road, Milton

Mean and standard deviation of the maximum deflection and SNC values are shown in Table 5.8. Overall, pre-flood mean maximum deflection was lower than post-flood mean maximum deflection. Pre-flood mean SNC was higher than post-flood mean SNC. Although there was an improvement in mean maximum deflection and SNC values in 2013 for both wheel paths, the mean values decreased again in 2014. This means that some sections of the pavement had deteriorated again. Mean deflection and SNC values improved in 2016.

Table 5.8: Mean and standard deviation of Maximum Deflection (D_0) and SNC of Haig Road, Milton

Name of Road, Wheel Path and Lane	**Sample Size (total Number of Chainage)**	**Parameter**	**D_0 (19 Dec 2010)**	**D_0 (3 April 2011)**	**D_0 (26 Mar 2013)**	**D_0 (8 Dec 2014)**	**D_0 (22 Mar 2016)**	**SNC (19 Dec 2010)**	**SNC (3 April 2011)**	**SNC (26 Mar 2013)**	**SNC (8 Dec 2014)**	**SNC (22 Mar 2016)**
Haig Road, Milton	104	Mean	0.335	0.376				5.32	5.10			
		Standard Deviation	0.221	0.255				1.985	2.080			
Haig Road, Milton	52	Mean			0.263	0.408	0.282			5.96	4.48	5.67
		Standard Deviation			0.151	0.240	0.168			2.091	1.462	1.924
1L-Outer WP	26	Mean	0.311	0.326	0.249	0.302	0.276	5.75	5.46	6.05	5.25	5.69
		Standard Deviation	0.246	0.236	0.138	0.166	0.167	2.292	2.158	2.118	1.593	1.970
1R-Inner WP	26	Mean	0.323	0.322	0.278	0.338	0.289	5.62	5.87	5.86	5.19	5.64
		Standard Deviation	0.240	0.252	0.165	0.210	0.172	2.140	2.411	2.101	1.840	1.915
2L-Inner WP	26	Mean	0.276	0.391				5.62	4.79			
		Standard Deviation	0.150	0.239				1.666	1.773			
2R-Outer WP	26	Mean	0.429	0.476				4.31	4.36			
		Standard Deviation	0.218	0.288				1.479	1.839			
Note: Lane 2 of Haig Road Milton was tested only in 2010 and 2011, As sample size varies from 2013 to 2016, separate entries have been included.												

FWD testing was conducted on Haig Road in Milton five times between 2011 and 2014. It shows that the rehabilitation programme after the flood contributed to the strength regain of the pavement. These results highlight the importance of long term observation of pavement is vital in modelling deterioration of pavement subjected to flooding.

5.5 HAIG ROAD IN AUCHENFLOWER

Most of the existing surface of Haig Road in Auchenflower is believed to be 28-year old asphalt (since 1988) in non-reconstructed areas. This section of Haig Road was partially inundated during the January 2011 flooding. It is also an industrial access road. The main difference between Haig Road in Milton and Auchenflower is that the Auchenflower section has thin AC layer with Gravel base and the Milton section has AC with CTB. This street was rehabilitated in 2011 after the flood. The data for this street were used to compare the pavement which has AC with CTB and pavements with thin AC and Gravel base (see Section 5.9). This road did not have any pre-flood data. Due to the similarity with Haig Road in Milton, only graphical representation and comparison of the maximum deflection and SNC of this road (Figures 5.22 to 5.25) were included in the analysis. These graphs show that deflection and SNC values improved in 2013 in comparison to the data collected immediately after flood. These results imply that many pavement sections regained strength after rehabilitation.

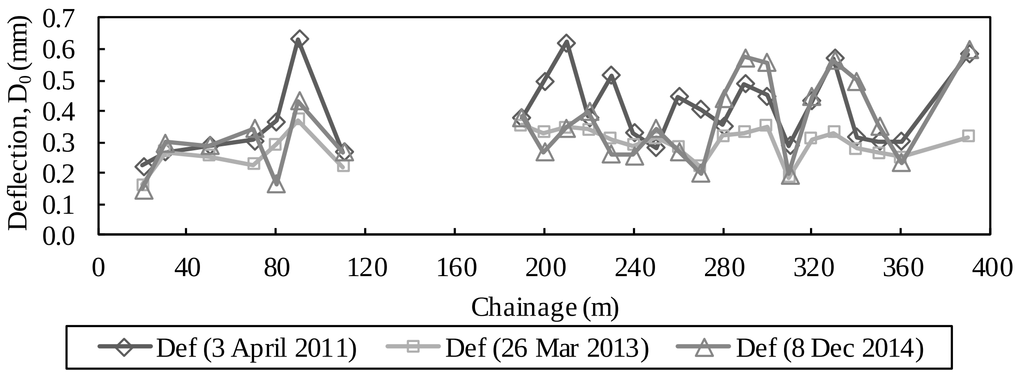


Figure 5.22: Maximum deflection vs chainage (outer wheel path [1L]) of Haig Road, Auchenflower

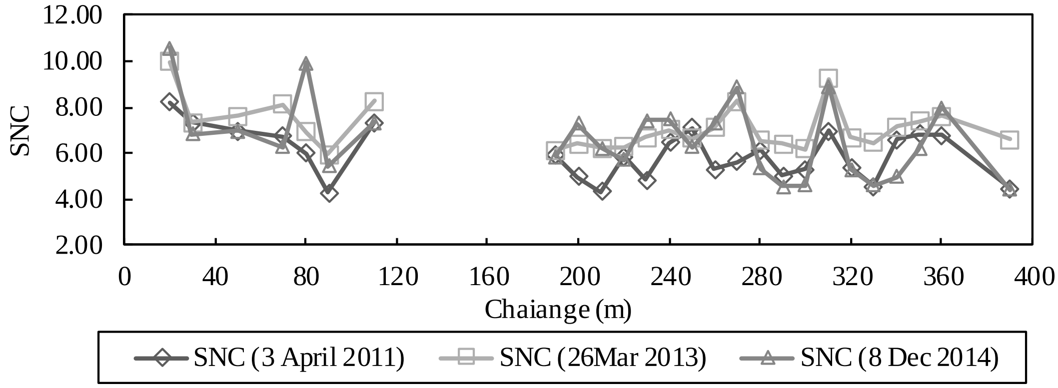


Figure 5.23: Modified structural number vs chainage (outer wheel path [1L]) of Haig Road, Auchenflower

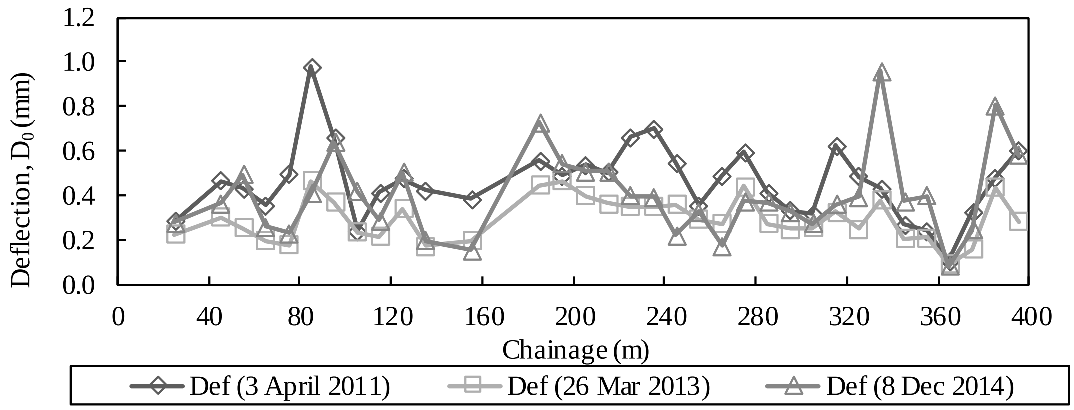


Figure 5.24: Maximum deflection vs chainage (inner wheel path [1R]) of Haig Road, Auchenflower

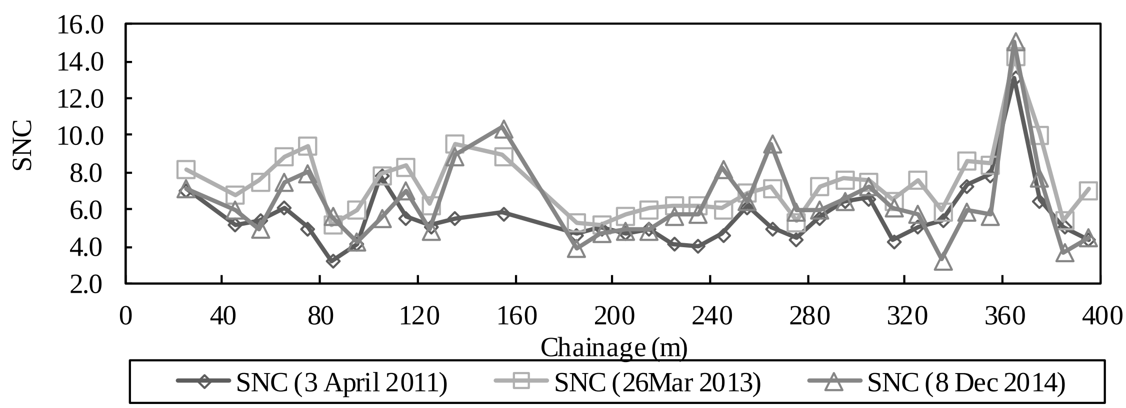


Figure 5.25: Modified structural number vs chainage (inner wheel path [1R]) of Haig Road, Auchenflower

5.6 SUTTON STREET IN CHELMER

Sutton Street in Chelmer (from Longman Terrace to End [cul-de-sac]) is one of the areas that was partially affected by flood and is a good example for comparing the changes in inundated and non-inundated sections of road. This street was included in the FWD testing program of Brisbane City Council. The most important point for this investigation is that all sections of this street were made at the same time and all sections are the same type (except 2R wheel-path that was patched in some areas).

The existing surfacing was built in 1975. Between chainage 16 and 63 is heavily patched and the patches over a storm water pipe have severe depression. The topography slopes slightly down from Rosebery Terrace to a low point at the gullies (chainage 32) before climbing slightly-moderately to Queenscroft Street. Surface failures predominately consist of moderate longitudinal and swell/shrink cracks. This is more severe between chainage 64-115. Some weathering has also occurred. Depressions were present in this street but they have become more severe after the flood with even recent patching settling. The patching differs in age. Rideability is generally ‘fair’ but over the storm water trench is ‘poor’.

This street is a cul-de-sac (ESA 1.5×10^4). Due to the traffic density and street function, it is supposed that the thickness of asphalt layer of 50 mm is normal for this kind of street. Pit sampling and Asphalt Coring (undertaken in December 2010) indicate two sections for the pavement as shown in Table 5.9.

Table 5.9: Thickness of surface layer (Sutton Street, Chelmer)

Ch. 0-65 m	**Ch. 65-133 m**
45-65 mm AC in Original Pavement (65-145 mm in Patched area)	40-60 mm AC in Original Pavement
135-150 mm Gravel with Sand & Silt	190-200 mm Gravel with Sand & Clay
170 mm Clayey Gravel with Sand	
370 – 400 mm (Total thickness)	240 – 250 mm (Total thickness)

Subgrade is classified as clay (with/without sand) in all sections and moisture contents are above the plastic limit that makes the pavement sensitive to the moisture.

5.6.1 Analysis of Deflection Data

In Sutton Street, chainage 0 m to 120 m was non-flood affected and chainage 120 m to 270 m was flood affected. This street did not have any pre-flood data. The graphical representations of deflection versus chainage (Figure 5.26) and strength versus chainage (Figure 5.27) indicate that deflection values were lower and modified structural number was higher in the pavement section that was not flooded when compared to the section that was flooded.

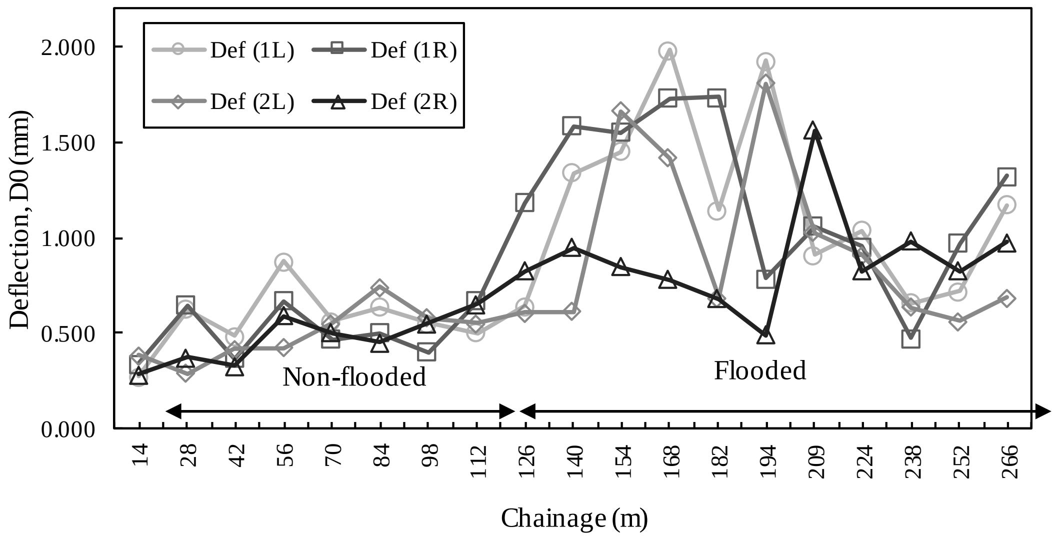


Figure 5.26: Maximum deflection at different chainages on Sutton Street, Chelmer

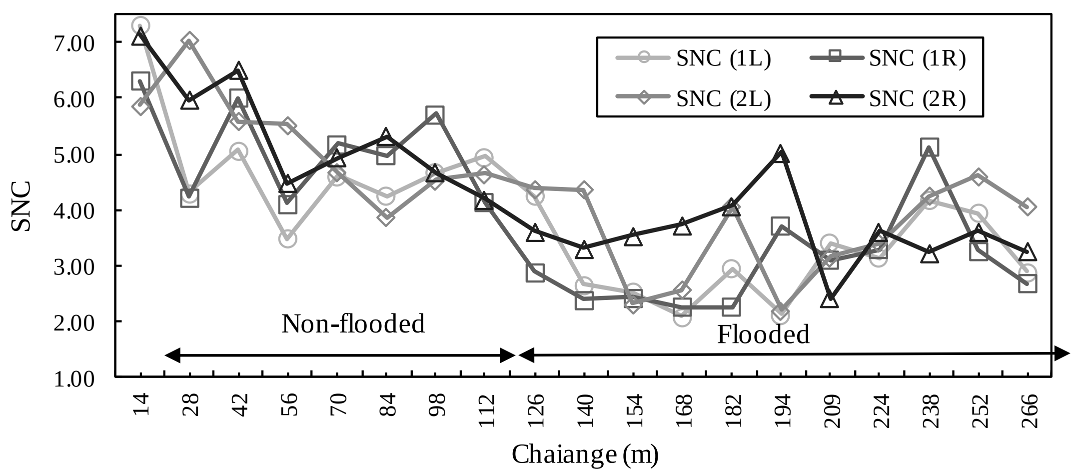


Figure 5.27: Modified structural number at different chainages on Sutton Street, Chelmer

Mean maximum deflection and SNC in Table 5.10 also indicate a clear difference in deflection and SNC values between flooded and non-flooded sections of both lanes. Mean maximum deflection and SNC of non-flooded section were lower than the flooded section.

Table 5.10: Mean and Standard deviation of Maximum Deflection (D_0) and SNC of Sutton Street, Chelmer

	Lane	1L		1R		2L		2R	
	Sample Size	9	10	8	11	9	10	8	11
Parameter		Non flooded	Flooded	Non flooded	Flooded	Non flooded	Flooded	Non flooded	Flooded
Max Deflection (mm)	Mean	0.570	1.233	0.506	1.213	0.503	1.000	0.465	0.885
	STD	0.161	0.454	0.137	0.411	0.139	0.465	0.131	0.265
SNC	Mean	4.77	2.99	5.08	3.04	5.12	3.50	5.39	3.59
	STD	1.061	0.700	0.876	0.842	0.972	0.896	1.040	0.634

By comparing the flood affected and non-flood affected pavement section of Sutton Street, Chelmer, it can be seen that the flood contributed to the decreasing strength of pavement. Inundation was the main cause of the rapid deterioration in the flood affected part. If the non-flooded section had some cracks or weakened surface condition before heavy rainfall and flood, moisture may have also infiltrated through them.

5.7 ALDERSGATE STREET IN OXLEY

Aldersgate Street was partially inundated during the January 2011 flooding. Brisbane City Council conducted an investigation of the pavement inundated by the flood to determine the impacts on the pavement condition and life of Aldersgate Street (Thornburgh Street to Wilpowell Street). Aldersgate Street was resurfaced with AC in 1973 and is a residential collector with ESA 3.7×10^4. Due to the traffic density and street function, it is supposed that the thickness of asphalt layer of 50 mm is normal for this type of street. To determine the effect of flooding on the pavement, FWD testing was undertaken in both wheel paths after the January 2011 flood.

5.7.1 *Analysis of Deflection Data*

Aldersgate Street, Oxley was first tested after flooding on 13 April, 2011, two years post-flood on 1 April, 2013 and almost four years post-flood on 8 December, 2014. Chainage 10 m to 105 m (Thornburgh Street) was not flooded while Chainage 105 m to Chainage 380 m (Wilpowell Street) was flooded. Deflection data collected two years after flooding shows a decrease in maximum deflection (D_0) and increases in SNC values in many chainages (Figures 5.28 to 5.31). Aldersgate Street was resurfaced on 25 May, 2011. It was selective reconstruction (patches) with 25-mm overlay and most of the repairs being in areas where there was flooding. Therefore, it can be concluded that both resurfacing and post-flood dry weather periods contributed to the strength regain of the flooded pavement section.

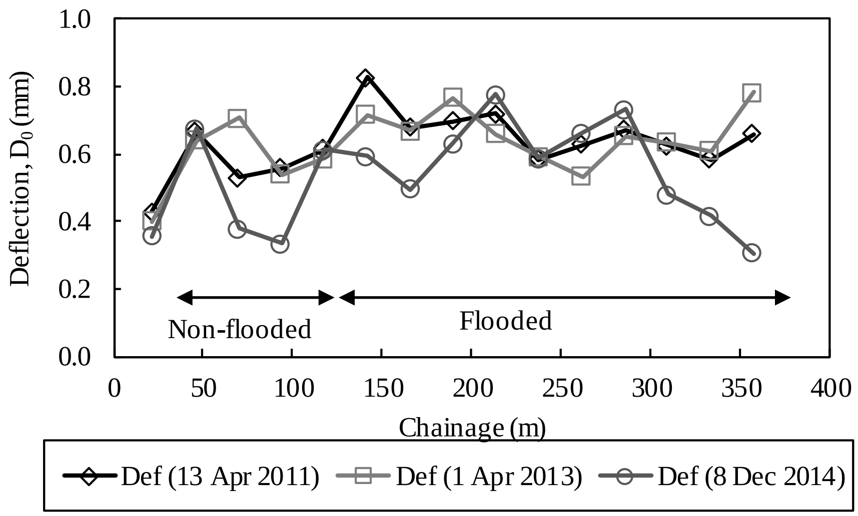


Figure 5.28: Maximum deflection at different chainages (outer wheel path [2R]) of Aldersgate Street, Oxley

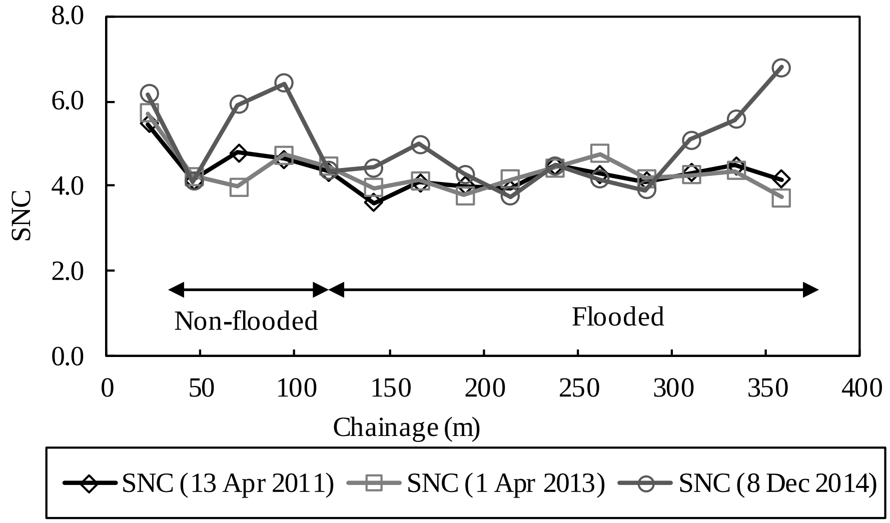


Figure 5.29: SNC at different chainages (outer wheel path [2R]) of Aldersgate Street, Oxley

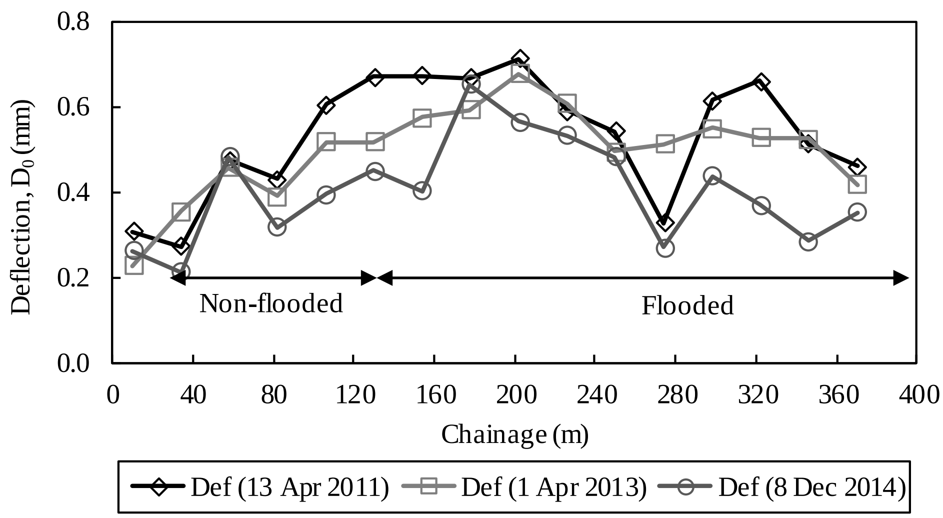


Figure 5.30: Maximum deflection at different chainages (inner wheel path [2L]) of Aldersgate Street, Oxley

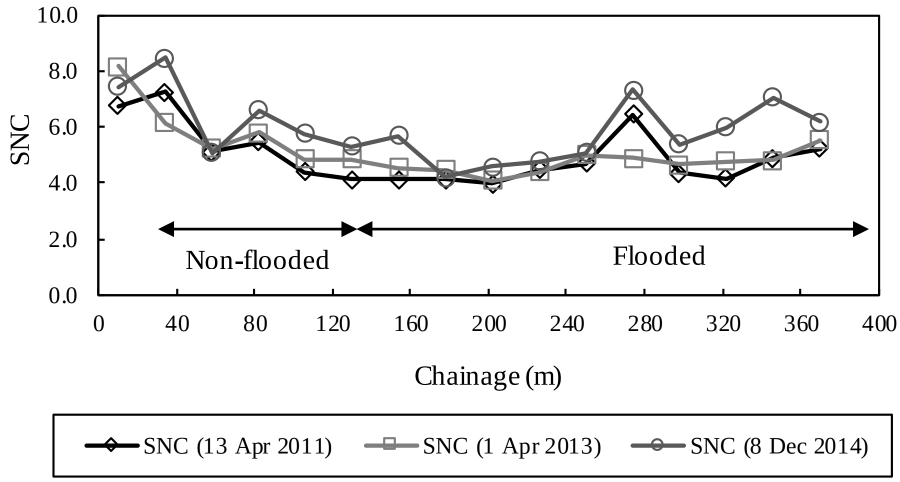


Figure 5.31: SNC at different chainages (inner wheel path [2L]) of Aldersgate Street, Oxley

Table 5.11 compares the mean maximum deflection and SNC in flooded and non-flooded areas of Aldersgate Street, Oxley. It indicates that mean maximum deflection was lower and SNC was higher in the non-flooded section compared to the flooded section of both wheel paths of Lane 2 of Aldersgate Street, Oxley.

Table 5.11: Comparison of mean maximum deflection and standard deviation of Aldersgate Street, Oxley

Date of Data Collection			**13-Apr-11**		**1-Apr-13**		**8-Dec-14**	
Sample Size (2L/2R)			**5**	**11/10**	**5**	**11/10**	**5**	**11/10**
Lane and Wheelpath	**Parameter**		**Non-Flooded**	**Flooded**	**Non-Flooded**	**Flooded**	**Non-Flooded**	**Flooded**
Inner Wheelpath (2L)	Maximum Deflection	Mean	0.418	0.584	0.389	0.544	0.334	0.435
		STD	0.134	0.115	0.111	0.067	0.107	0.117
	SNC	Mean	5.79	4.60	6.04	4.73	6.66	5.60
		STD	1.181	0.727	1.291	0.376	1.349	0.986
Outer Wheelpath (2R)	Maximum Deflection	Mean	0.559	0.667	0.574	0.662	0.470	0.567
		STD	0.090	0.072	0.116	0.077	0.160	0.145
	SNC	Mean	4.67	4.15	4.63	4.18	5.39	4.74
		STD	0.509	0.267	0.664	0.307	1.073	0.906

The percentage of post-flood strength gain in flooded and non-flooded part of the pavement are compared in Figures 5.32 and 5.33. Percentage of strength loss 2 years post-flooding is lower in the non-flooded section than the flooded section in inner wheel path (2L). For outer wheel path (2R), the strength gain 2 years post-flooding is higher in the flooded section than the non-flooded section.

Among the 16 testing points in inner wheelpath and 15 testing points in outer wheelpath, 5 testing points in both wheel paths were in the non-flooded section. In the non-flooded zone, data collected 2 years post-flood indicated that four testing points in both lanes gained strength, e.g. 2.4% to 21% in 2L and 2.2% to 4.4% in 2R. In the flood affected zone, strength gain varied from 3.2% to 17.8% (8 sections) in the inner wheel path and 1% to 10.5% (5 sections) in the outer wheel path.

In the non-flooded zone, data collected 4 years post-flood indicated that four testing points in both lanes gained strength, e.g. 10% to 31% in 2L and 0.5% to 38.5% in 2R after the 2011 flooding event. In the flood affected zone, strength gain varied from 1.6% to 45% (all sections) in the inner wheel path and reduction in strength was 6.7% to 63% (6 sections) in the outer wheel path.

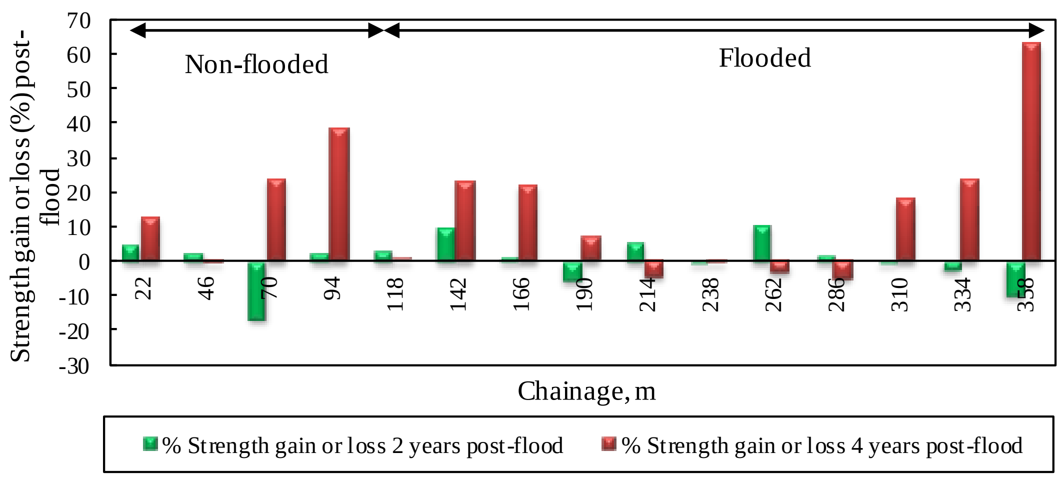


Figure 5.32: Post-flooding strength gain of Aldersgate Street, Oxley (outer wheel path [2R])

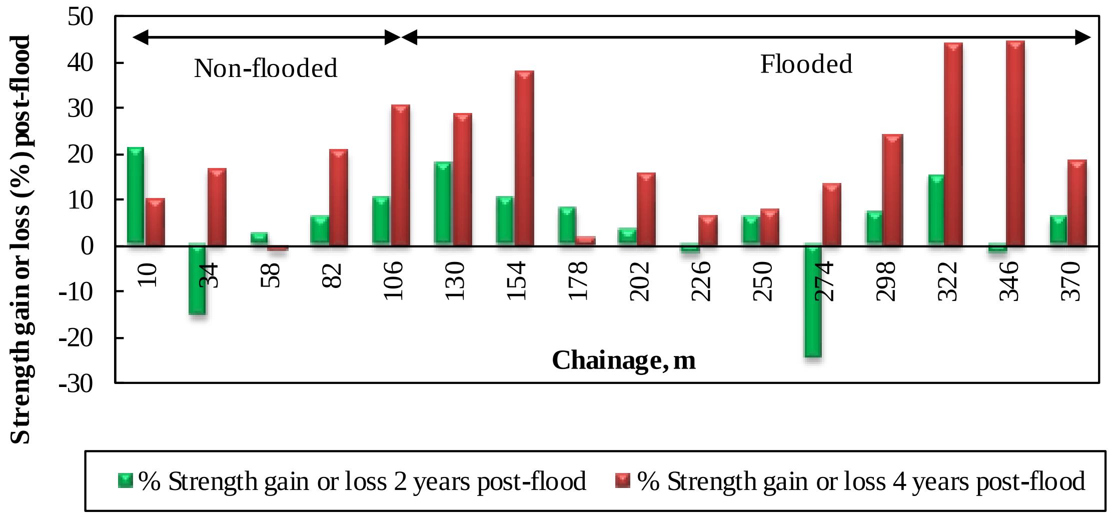


Figure 5.33: Post flooding strength gain of Aldersgate Street, Oxley (inner wheel path [2L])

5.8 CORDELIA STREET, SOUTH BRISBANE - A CASE STUDY

Brisbane City Council conducted FWD testing on Cordelia Street, South Brisbane (Glenelg Street to Montague Road). Cordelia Street is an arterial road that was partially inundated during the January 2011 flooding. This study conducted a case study on this street using the structural and surface condition data. To determine the effect of flooding on the pavement, FWD testing was undertaken in both wheel paths after the January 2011 flood. Records indicate that Cordelia Street (from Glenelg Street to Peel Street) was reconstructed with asphalt in 2010 with the previous asphalt resurfacing being done in 1982. The section between Peel Street and Boundary Street was last resurfaced in 2002. The existing surface is believed to be 1-year and 9-year old asphalt. Pit sampling and asphalt coring (prior to the 2010 reconstruction) indicated three different pavement profile sections. Subgrade in these sections was classified as Clayey Gravel. Layer thickness and subgrade CBR for the different sections of the pavement is shown in Table 5.12.

Table 5.12: Layer thickness of Cordelia Street, South Brisbane

Section	**Road name**	**Chainage (m)**	**Thickness (mm)**	**Layer**	**Subgrade CBR (%)**
Section 1	Glenelg St to Melbourne St	400-870	190	AC (in inner lanes, reconstructed area)	
			180	Gravel with Sand & Silt	
Section 2	Melbourne St to Peel St	870-1096	190	AC	2.5 to 7
			130-180	Gravel with Silt / Clay	
			60-240	Clayey Gravel	
Section 3	Peel St to Boundary St	1096-1285	60-130	AC	
			120-220	Gravel with Silt / Clay	2 to 16

5.8.1 Analysis of Deflection Data

FWD data were collected in 2011, 2013, 2014 and 2016. Deterioration of structural condition in the inner and outer wheel paths of Cordelia Street immediately after the January 2011 flood, in 2013, 2014 and 2016 are presented in Figures 5.34 to 5.37. A comparison of the 2011 and 2013 data indicates that pavement sections had lower maximum deflection (D_0) and higher SNC values in 2013 than in 2011. However, many chainage points between 400 m and 850 m had higher maximum deflection and lower SNC values in 2014. Further investigation of these chainages indicates that some sections of the road had loss of surface condition. As a result of the deterioration, part of the pavement section was rehabilitated again in 2015.

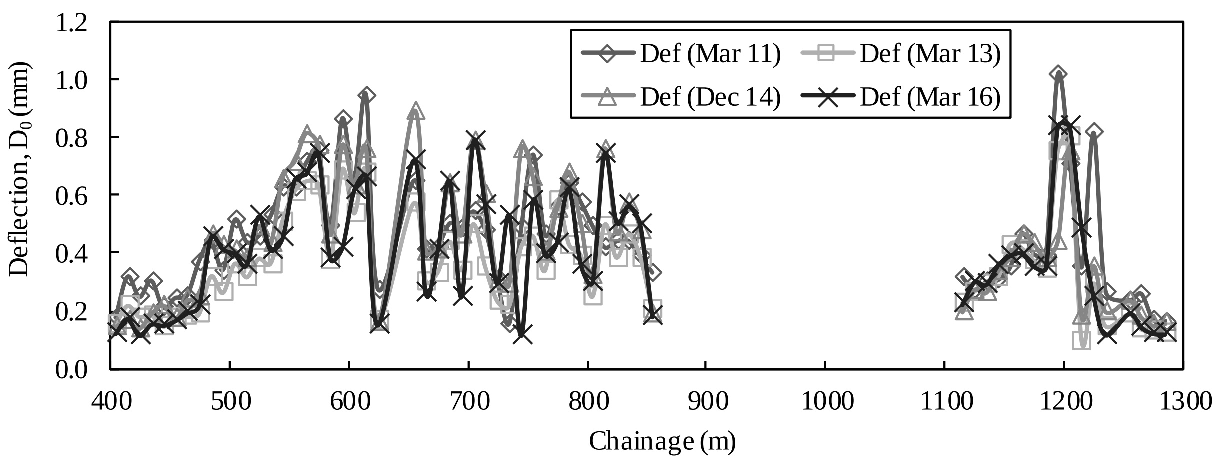


Figure 5.34: Maximum deflection at different chainages (inner wheel path [1R]) of Cordelia Street, South Brisbane

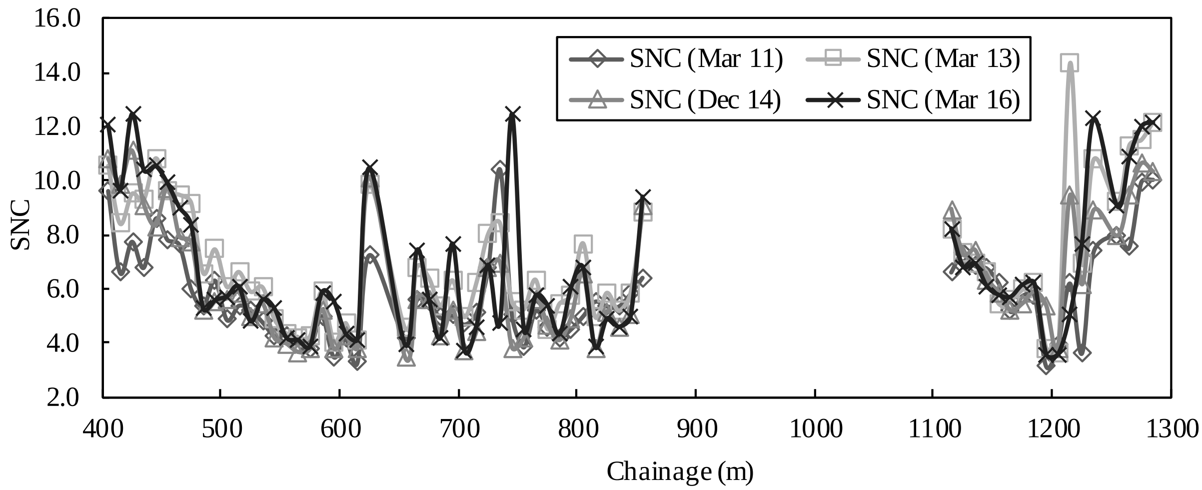


Figure 5.35: SNC at different chainages (inner wheel path [1R]) of Cordelia Street, South Brisbane

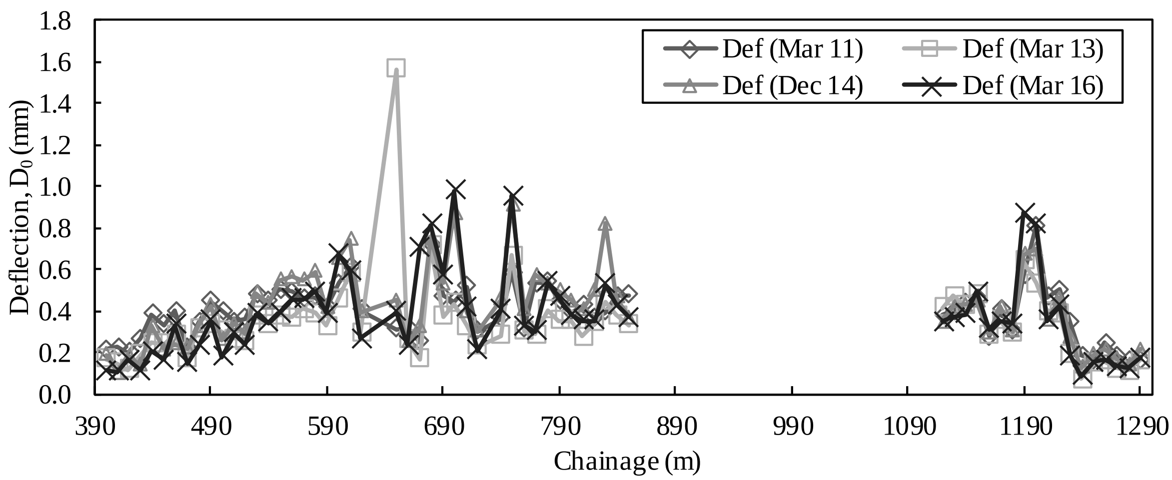


Figure 5.36: Maximum deflection at different chainages (outer wheel path [1L]) of Cordelia Street, South Brisbane

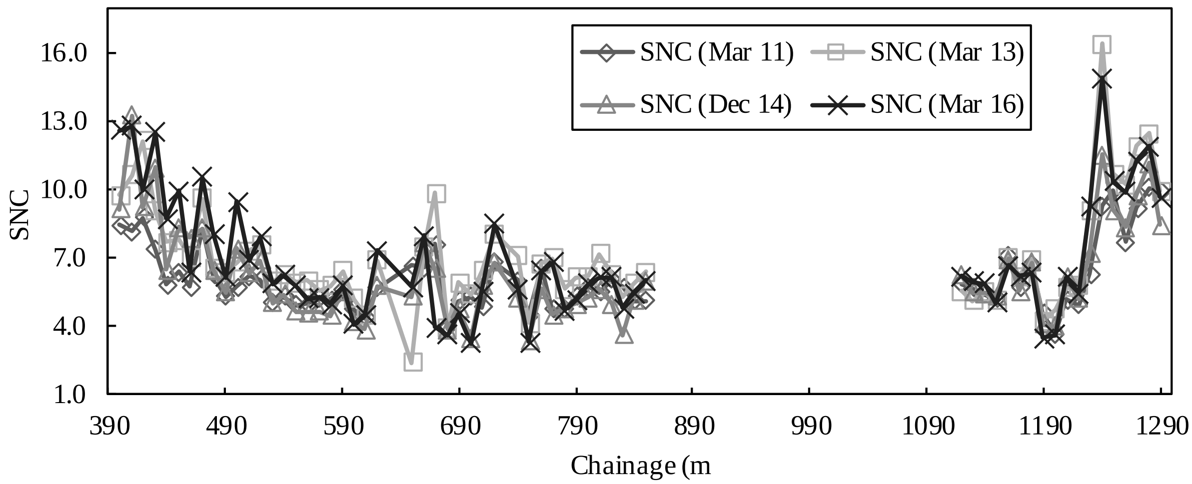


Figure 5.37: SNC at different chainages (outer wheel path [1L]) of Cordelia Street, South Brisbane

The changes in post-flood strength over the last six years at different chainages of Cordelia Street are shown in Figures 5.38 and 5.39. Two years post-flood data indicates that most of the pavement sections gained some strength, ranging from 1.2% to 131% for the inner wheel path and 0.25% to 76.4% for the outer wheel path. Three years post-flood data indicates that strength gain varied from 0.68% to 69.3% for the inner wheel path and 0.28% to 62.6% for the outer wheel path. However, more and more sections started to lose strength 3 years post-flood compared to two years post-flood. Five years post-flood (data collected in 2016) showed an improvement in the strength gain due to the rehabilitation works conducted in 2015.

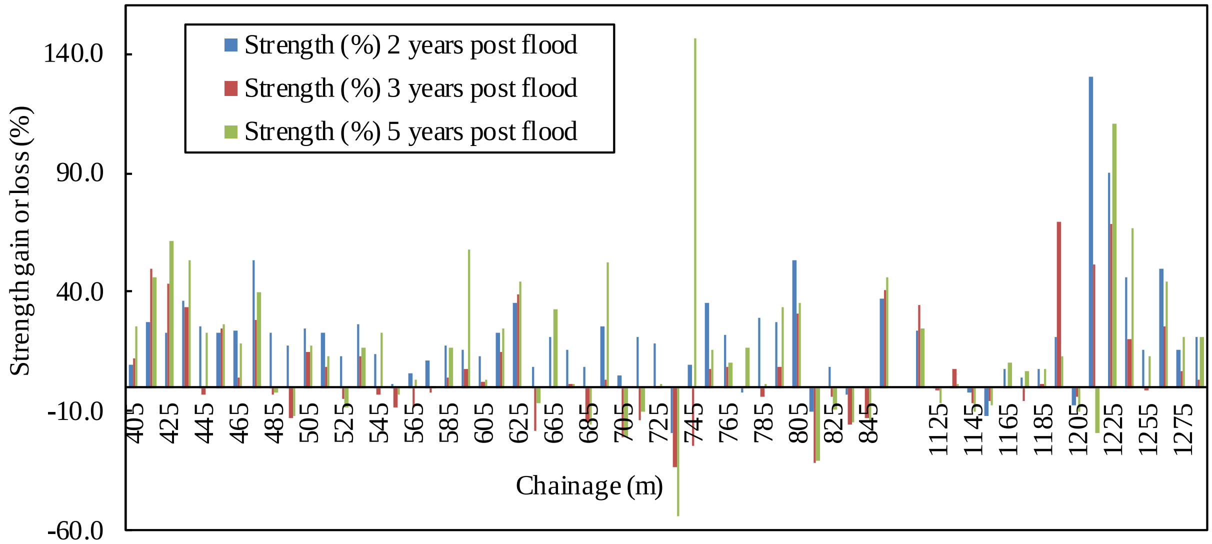


Figure 5.38: Post-flooding strength gain at different chainages (inner wheel path [1R]) of Cordelia Street, South Brisbane

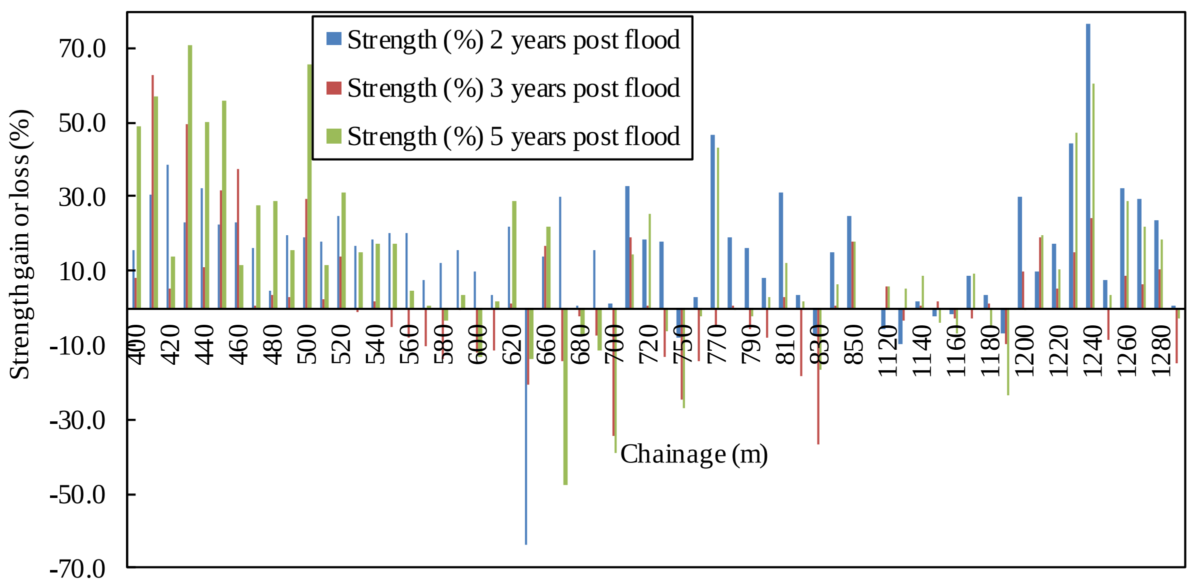


Figure 5.39: Post-flooding strength gain at different chainages (outer wheel path [1L]) of Cordelia Street, South Brisbane

Tables 5.13 and 5.14 compare data from Cordelia Street (N = 125) collected in 2011, 2013 and 2014 using the paired sample t-tests. The mean maximum deflection value was significantly higher, and the mean SNC lower in 2011 compared 2013 ($p < 0.005$, 95% Confidence interval). The mean maximum deflection value was significantly lower, and the mean SNC was higher in 2013 than 2014 ($p < 0.005$, 95% Confidence interval). In contrast, the mean maximum deflection value were similar in 2011 and 2014 ($p > 0.05$, 95% Confidence interval). However, mean SNC was higher in 2014 than 2011 ($p < 0.005$, 95% Confidence interval). These results suggest that pavements suffered from losses of structural strength immediately after the flooding while post-flood restoration works helped pavements regain structural strength.

Table 5.13: Paired Samples t-tests (Maximum Deflection) of Cordelia Street, South Brisbane

Pair	Variable	Mean	N	Std. Deviation	Std. Error Mean	t	p-value
Pair 1	Deflection 2011	.4224	122	.168514	.015257	5.336	.000
	Deflection 2013	.3509	122	.189495	.017156		
Pair 2	Deflection 2013	.3509	122	.189495	.017156	-5.332	.000
	Deflection 2014	.4220	122	.199881	.018096		
Pair 3	Deflection 2011	.4224	122	.168514	.015257	.032	.975
	Deflection 2014	.4220	122	.199881	.018096		

Table 5.14: Paired Samples t-test (SNC) of Cordelia Street, South Brisbane

Pair	Variable	Mean	N	Std. Deviation	Std. Error Mean	t	p-value
Pair 1	SNC 2011	5.9847	122	1.60592	.14539	-8.529	.000
	SNC 2013	7.0450	122	2.36838	.21442		
Pair 2	SNC 2013	7.0450	122	2.36838	.21442	7.547	.000
	SNC 2014	6.2807	122	2.15874	.19544		
Pair 3	SNC 2011	5.9847	122	1.60592	.14539	-2.645	.009
	SNC 2014	6.2807	122	2.15874	.19544		

5.8.2 Analysis of Surface condition Deterioration

Further investigation was conducted on the road surface inventory system data of Cordelia Street (Table 5.15). A comparison between data collected in 2011 and 2015 showed that there was an increase in surface condition failures in 2015. These failures included: crocodile cracking, seepage, ravelling, swelling, rutting, AC patching, in between chainages 0 m - 227 m, 355 m – 461 m, 473 m – 667 m, and 686 m – 1180 m. These results also match the FWD deflection data analysis. Increases in surface condition distress were observed in many pavement sections following the January 2011 flooding. Photographs in Figure 5.40 show the escalation of the surface condition deterioration in some sections of the road. Figure 5.41 shows the pavement repairs undertaken in the form of AC patching as a result of the accelerated deterioration.

Table 5.15: Deterioration of surface condition from visual inspection data

Inspection Date	Section	Classified Condition	Pavement Failures	Surface Failures
13/02/2003	Glenelg to Russell	F	1.6%	0.1%
	Russell to Melbourne	C	0.0%	0.1%
	Melbourne to Peel	F	1.6%	0.4%
	Peel to Boundary	A	0.0%	0.0%
2011	Glenelg to Russell			
	Russell to Melbourne	Recent reconstruction		
	Melbourne to Peel			
	Peel to Boundary	F	2.7%	2.9%

Another investigation into the repairs undertaken for different sections of Cordelia Street showed that several sections needed to be repaired four times in the five years after the flood. A small section of the pavement was repaired on September 11, 2011; a large section of the pavement was repaired prior to September 2, 2012; parts of the pavement were repaired on 15 October 2014 and 31 May 2015 (Figures 5.42 and 5.43).

Figure 5.40: Deterioration of surface condition and pavement repair of flood affected parts of Cordelia Street, South Brisbane (pictures taken in 2015)

Figure 5.41: Pavement repairs due to surface condition deterioration of flood affected parts of Cordelia Street, South Brisbane

Figure 5.42: Location of flooding on 13 January, 2011 (yellow sections) and small pavement repair by 11 September, 2011 (Aerial image)

Figure 5.43: Large pavement repairs by 2 September, 2012, additional pavement repairs by 15 October, 2014, pavement repairs by 31 May, 2015 (Aerial image)

Deterioration of surface condition, such as roughness and rutting, may not always be visible immediately after flooding. Flood-affected pavements with reduced structural and subgrade strength are more prone to loss of surface condition.

5.9 RMS NSW ROAD

Due to lack of information on pavement history, FWD deflection data of RMS NSW roads were not used for detailed analysis. Figure 5.44 compares the pre- and post-flood maximum deflection of Sturt Highway from the RMS NSW data. This figure shows some sections had pre-flood maximum deflections greater than post-flood deflections while other sections had exactly the opposite. Pre-flood mean maximum deflection of this road was 0.795 mm and post-flood mean maximum deflection of this road was 0.676 mm.

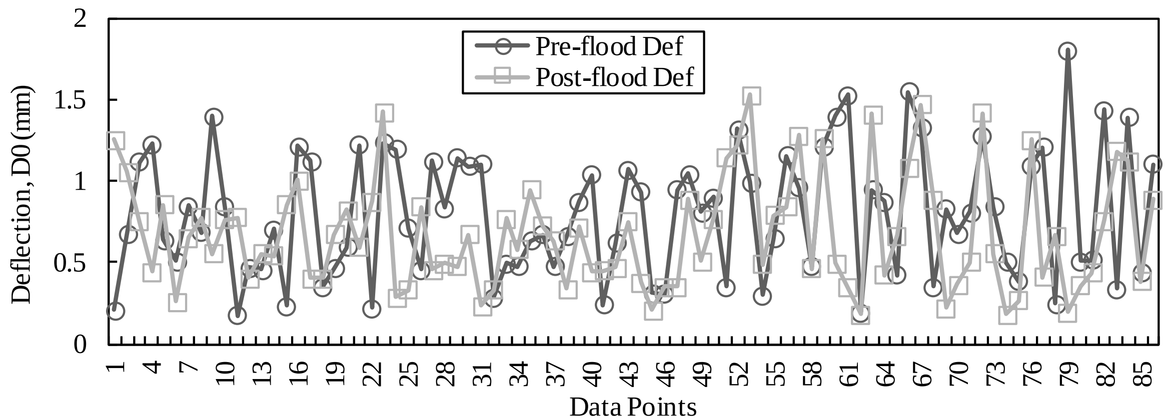


Figure 5.44: Pre- and post-flood deflection data of Sturt Highway, RMS NSW

5.10 IDENTIFYING THE TREND OF DETERIORATION

In some cases, field testing was done only after a flood. In these cases, a comparison of the trend of post-flood strength gain or loss was determined. An extensive analysis of the collected data exhibited three different combinations of deterioration trends for the pavements subjected to flooding. These trends are presented in the form of schematic diagrams as three scenarios (Figures 5.45 to 5.47). These schematic diagrams are supported by field observations data in this section.

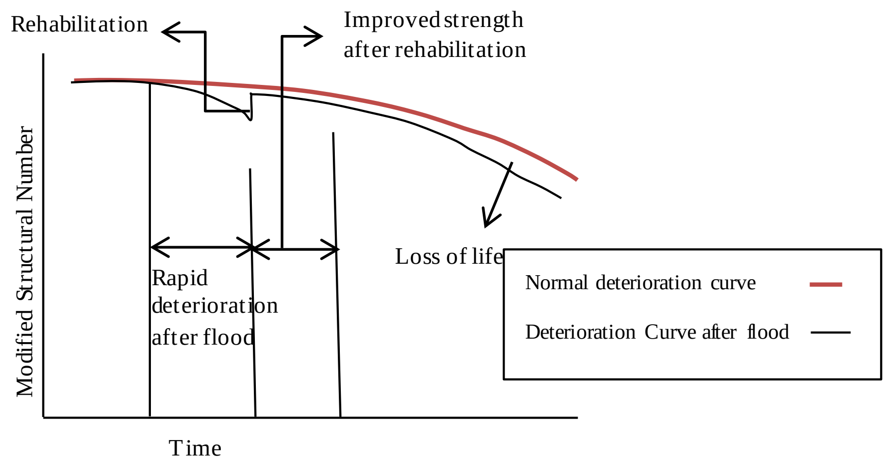


Figure 5.45: Scenario 1- Schematic diagram of the assumption that the road was rehabilitated after the flood and an incremental strength gain occurred due to rehabilitation; it did not regain the post-flood strength

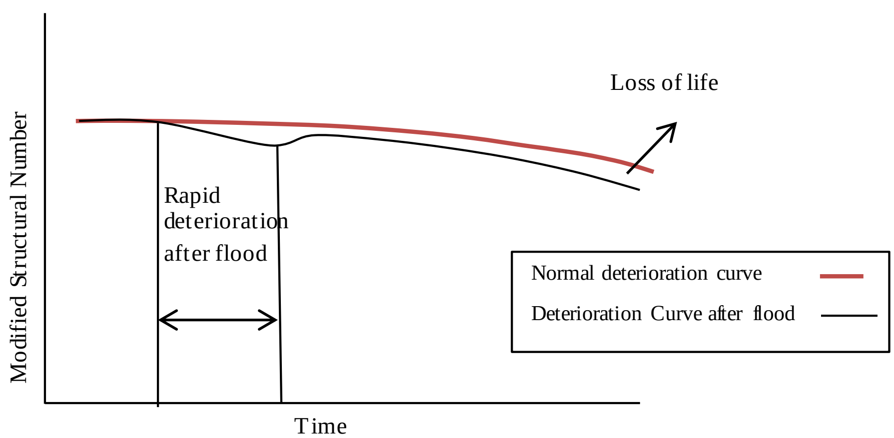


Figure 5.46: Scenario 2- Schematic diagram of the assumption that the road is regaining strength after the rapid deterioration phase without any post-flood rehabilitation; but it did not regained the pre-flood strength

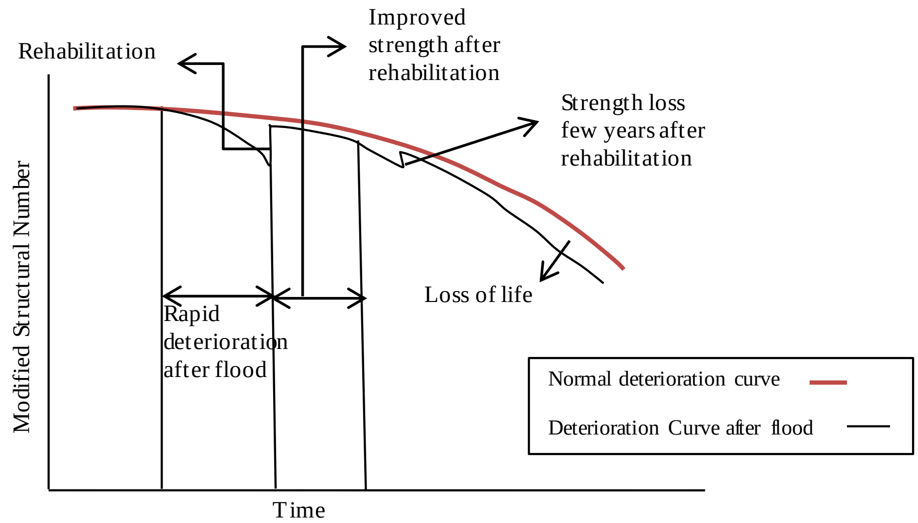


Figure 5.47: Scenario 3- Schematic diagram of the assumption that the road was rehabilitated after flood, regained incremental strength but started losing strength again and may need to be repaired again in future

In Figures 5.45 to 5.47, the red line shows the strength curve without the flooding event. Figure 5.45 represents the first deterioration trend where the pavement has rapidly deteriorated immediately after the flood. It was rehabilitated and is regaining incremental strength. Figure 5.46 illustrates the second deterioration trend where pavement is regaining incremental strength without any rehabilitation. Figure 5.47 represents the schematic diagram of the third deterioration trend where the pavement section has undergone rapid deterioration immediately after flooding, was rehabilitated and regained some strength. However, it continuously lost strength and may need to be repaired again in future. It should be noted that in all three cases, pavements failed to regain pre-flood strength, even after rehabilitation. However, the pavements could be fully restored to the strength they had pre-flooding if the rehabilitation involved a greater re-sheet or overlay thickness. Also, the pavement may have lost some service life due to the rapid deterioration caused by the flood.

Further assessment was conducted on specific pavement sections with very high deterioration rates. Figure 5.48 shows two sections of an AC pavement with pre- and post-flood data. The pavement was scheduled for rehabilitation before the flood. The dotted line on the diagram shows the predicted deterioration curve (with rehabilitation) of the pavement without the flooding event. Due to flooding, the pavement section lost strength quicker than expected.

The street was rehabilitated on 26 May 2011. As a result, the pavement section gained incremental structural strength. However, the section did not regain the pre-flood strength as expected after the rehabilitation. The dotted line shows the pavement deterioration curve (with rehabilitation) without the flooding event. The section would be performing well after the flood if there was no flooding. The second section in Figure 5.48 also lost strength immediately after the flood. However, 4 years post-flood testing indicates that it regained more strength (24%) compared to the pre-flood strength following rehabilitation and the dry weather period.

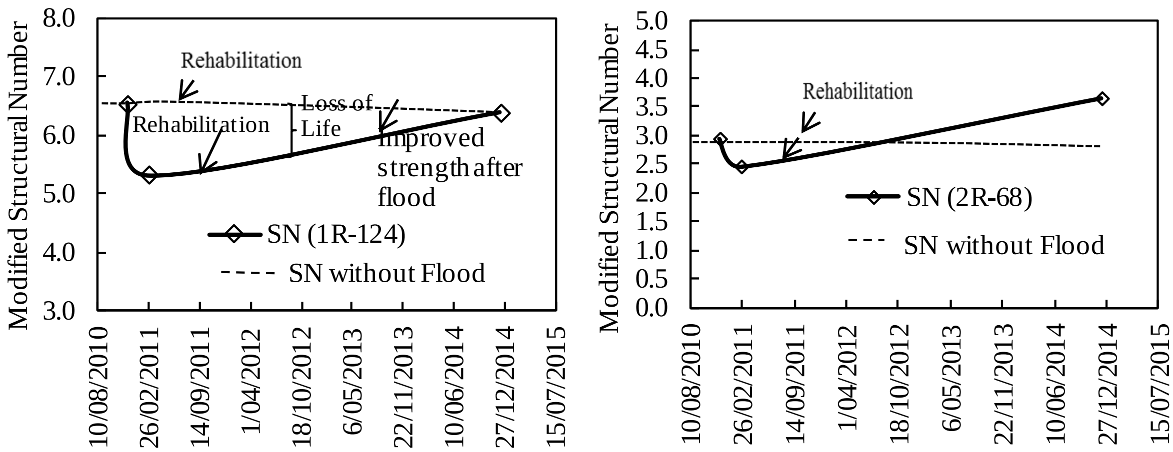


Figure 5.48. Comparison of deterioration of structural strength for a lightly trafficked street (Scenario 1)

Figure 5.49 illustrates two AC pavement sections of an industrial road. The dotted line shows the predicted strength curve of the pavement section without the flood event. The pavement lost 33% of its strength immediately after the flood. The section was resurfaced in May 2011. The 2013 post-flood data indicated an improvement in strength, which is possibly the contribution of both resurfacing and the dry weather period post-flood. However, the 2014 data indicated the pavement had lost 5% of its strength compared to the 2013 strength data. These findings warrant the need for further investigation to identify the causes of deterioration.

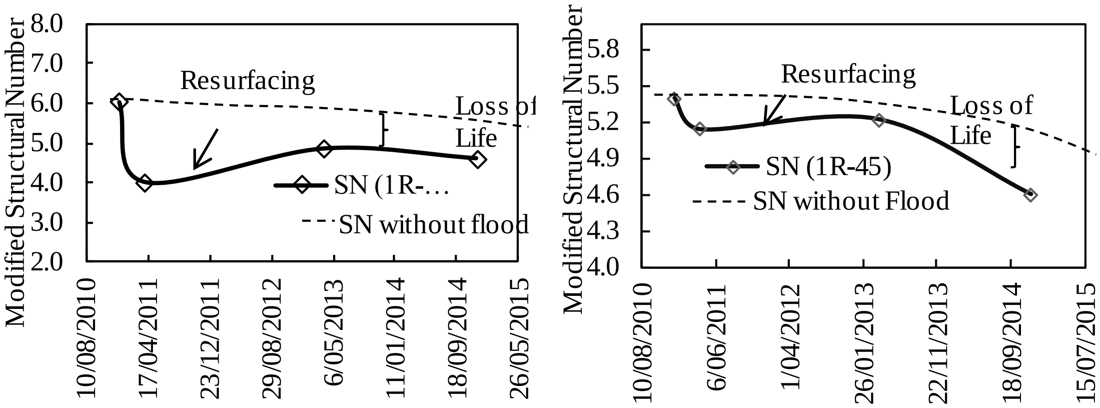


Figure 5.49. Comparison of deterioration of structural strength of an industrial road (Scenario 1)

The pavement sections illustrated in Figures 5.50 and 5.51 do not have pre-flood data but have post-flood data collected in 2011, 2013 and 2014. Figure 5.50 compares the flooded and non-flooded section of a lightly trafficked local road. Pavement gained strength after the flood in the non-flooded section during the dry weather period. The flooded section of Figure 5.50 re-gained strength, two years after the flooding. However, the 4 years post-flood data indicates that the section is again losing strength. This indicates a possible problem with the subgrade which may be affected by inundation.

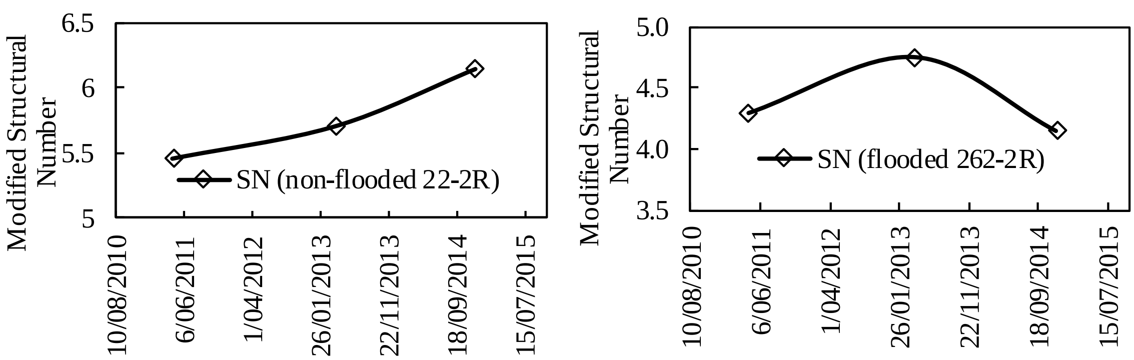


Figure 5.50. Strength curve of non-flooded and flooded section of a lightly trafficked road without rehabilitation (Scenario 2)

Figure 5.51 shows a section of an arterial road which did not regain strength after minor patching. The second section of the same road has regained strength after minor patching but has started to deteriorate again and may need to be repaired in the future. As this is an arterial road, it may have deteriorated rapidly when the road was reopened for heavy traffic. The dotted line shows the predicted strength curve of the pavement with rehabilitation.

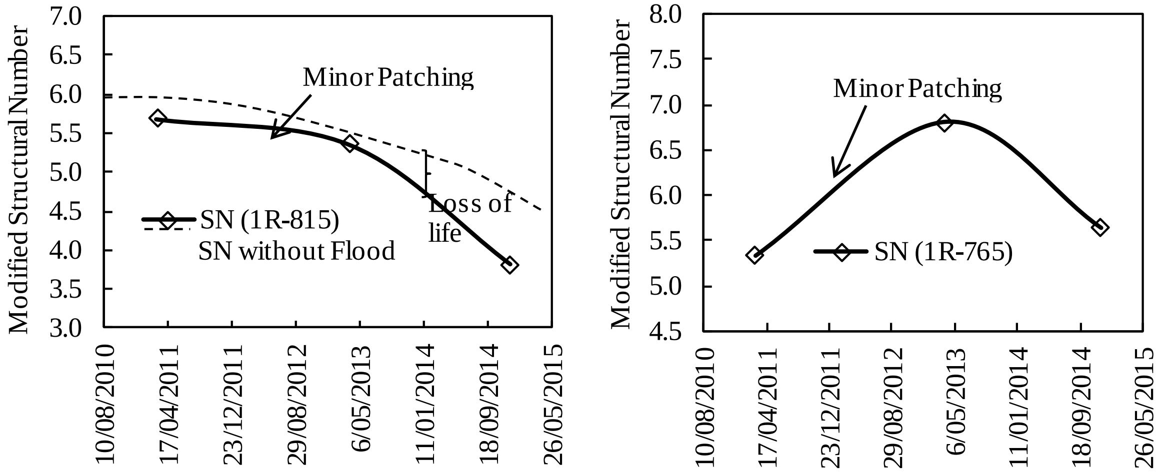


Figure 5.51. Comparison of deterioration of structural strength of an arterial road (Scenario 3)

The above analysis and results suggest that flood affected pavements deteriorate rapidly rather than gradually as expected by most available deterioration prediction models. The results suggest that further investigation should be conducted on the pavement sections which failed to gain strength during dry weather periods or even after rehabilitation. It indicates the possible problem of subgrade weakening which needs proper attention.

Pre-flood field observation data is available for only two flood affected roads in the Brisbane City Council area. After in-depth analysis of pre- and post-flood data, four distinctive trends of deterioration of individual sections of a flood affected local road pavement were revealed (Figure 5.52).

Trend-1 refers to the pavement section with post-flood deterioration (33% strength loss). Two years after the flood, it had an increase in strength (47%) due to rehabilitation and subsequent dry weather periods but did not restore to pre-flood strength. Moreover, 4 years post-flood data indicates that the section has started to deteriorate again. These pavement sections will need rehabilitation sooner than expected due to the damage caused by the rapid deterioration phase of flood. It will increase the cost of road maintenance.

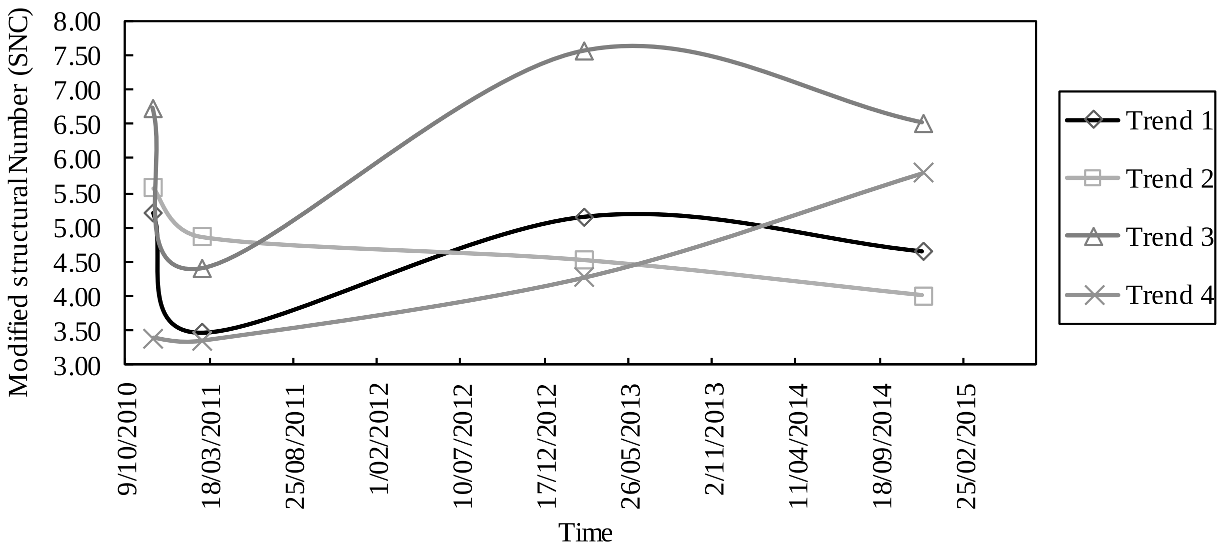


Figure 5.52. Deterioration trends for different sections of a flood affected pavement

Trend-2 indicates that the pavement section never restored to its pre-flood strength despite rehabilitation. A comparison of pre-flood data of the section with 4 years post-flood data indicates a loss of structural strength (28%). It will need to be repaired again in future. It indicates a possible problem with the subgrade which may be affected by inundation.

Trend-3 indicates that this pavement section regained its full pre-flood strength. An increase in strength (12%) was observed in 2013 due to the resurfacing work done in 2012 as well as the dry weather period. This section also deteriorated only slightly (3% loss of strength) after four years.

Trend-4 refers to an ideal condition with no significant loss of strength post-flood and strength gain observed after rehabilitation. This trend is not common in the observational data and this local road has a CTB with AC (AC layer thickness 50 mm). Further investigation of pavements with cement treated based is recommended for better understanding of the performance of such pavements in flood affected areas.

In the first two cases, the pavements failed to return to their pre-flood strengths and may have lost some service life due to rapid deterioration, even after rehabilitation. This could be due to the possible loss of subgrade strength, or rehabilitation which may not be always possible or feasible due to the road location, such as including a greater overlay thickness or re-sheeting. However, for the fourth case (Trend 4), the pavements were fully restored to their strength prior to the flooding. These findings warrant the need for further investigation of the pavement sections which failed to gain their strength during the dry weather period, or even after rehabilitation. The possible problem, or permanent possible weakening of the subgrade, appears to need greater attention.

5.11 THIN AC VS AC WITH CTB

Tables 5.16 and 5.17 compare Thin Asphalt Concrete (AC) pavements (thickness of asphalt layer less than 60 mm) with gravel base and AC pavements (thickness of asphalt layer 15-75 mm) with cement treated base (CTB) using independent samples t-tests. Thin Asphalt Concrete pavements were cul-de-sac (traffic volume 1.5×10^4), residential collectors (traffic volume 3.7×10^4) and industrial access roads (traffic volume 1.5×10^6). AC pavements with cement treated base were industrial access roads (traffic volume 1.5×10^6). There was a significant difference in mean maximum deflection and the SNC in 2010, 2011, 2013 and 2014 between Thin AC pavements with gravel base and AC pavements with CTB.

AC pavements with CTB performed significantly better than Thin AC pavements with gravel base after the January 2011 flood in the observational data of the Brisbane City Council area. This result should be taken into consideration when building resilience into future road pavements in flood prone areas. Although a detailed investigation on pavement materials is important, it was beyond the scope of this research.

Table 5.16: Comparison of Maximum Deflection of Thin AC with gravel base and AC with cement treated base (CTB)

Variable	**Pavement material**	**N**	**Mean**	**Std. Deviation**	**Std. Error Mean**	**F-value**	**t-value**	**p-value**	**Mean Difference**	**95% Confidence Interval of the Difference**	
										Lower	Upper
Deflection_2010	Thin AC-Gravel	59	1.057	0.486	0.063	32.126	12.976	.000	0.722	0.612	0.832
	AC(<75)&200CTB	104	0.335	0.221	0.022						
Deflection_2011	Thin AC-Gravel	227	0.884	0.533	0.035	22.287	9.398	.000	0.481	0.380	0.582
	AC(<75)&200CTB	122	0.403	0.257	0.023						
Deflection_2013	Thin AC-Gravel	31	0.562	0.123	0.022	.939	9.812	.000	0.299	0.238	0.359
	AC(<75)&200CTB	52	0.263	0.151	0.021						
Deflection_2014	Thin AC-Gravel	90	0.734	0.381	0.040	20.376	7.324	.000	0.414	0.302	0.525
	AC(<75)&200CTB	52	0.320	0.188	0.026						

Table 5.17: Comparison of Modified Structural Number of Thin AC with gravel base and AC with cement treated base (CTB)

Variable	**Pavement material**	**N**	**Mean**	**Std. Deviation**	**Std. Error Mean**	**F-value**	**t-value**	**p-value**	**Mean Difference**	**95% Confidence Interval of the Difference**	
										Lower	Upper
SNC_2010	Thin AC-Gravel	59	3.429	1.005	0.131	31.919	-6.849	.000	-1.896	-2.443	-1.349
	AC(<75)&200CTB	104	5.325	1.985	0.195						
SNC_2011	Thin AC-Gravel	227	3.996	1.350	0.090	45.465	-4.773	.000	-0.869	-1.227	-0.511
	AC(<75)&200CTB	122	4.865	2.035	0.184						
SNC_2013	Thin AC-Gravel	31	4.747	0.864	0.155	25.346	-3.063	.003	-1.210	-1.995	-0.424
	AC(<75)&200CTB	52	5.956	2.091	0.290						
SNC_2014	Thin AC-Gravel	90	4.419	1.388	0.146	2.625	-2.895	.005	-0.805	-1.357	-0.253
	AC(<75)&200CTB	52	5.224	1.704	0.236						

Note: N=Sample size

5.12 MODELLING DETERIORATION OF STRUCTURAL CONDITION (FIRST MODEL)

Accurate prediction of the post-flood short-term rapid deterioration phase of pavement is key to avoiding long term effects on road asset management. It would improve the decision making process of future rehabilitation and repair of flood-affected roads. This research developed a deterministic structural deterioration model to predict the rapid deterioration phase of the pavement impacted by river flooding, or the gradual rise of flood water. The observational data of local roads obtained from Brisbane City Council were used to develop the model. The data were collected following the flooding event in January 2011. The impacts of flash flooding were not considered for the model as the damage due to such events can be catastrophic and presents uncertainty in prediction.

Two models were developed, first model had one dependent variable, time. Later, after the completion of the Dynamic Cone Penetration Testing (DCP) of the pavement sections that were included in the modelling, another model was developed. This model expresses modified structural strength ratios (for rapid deterioration) as a function of time lapse in deflection measurement after flooding, subgrade strength as CBR (California bearing ratio) values and design traffic loading as Millions of Equivalent Standard Axles (MESA) values.

When developing the models, pavement category, thickness of the asphalt layer and traffic class were considered. The model used data collected during November/December 2010 (pre-flood) and February 2011 (post-flood). The model was developed using the observational data of the local roads followed by the January 2011 flood and collected within six weeks of the flood. After six weeks, the model may over predict the deterioration and, thus, become invalid. This is the short-term rapid deterioration phase when pavements are in their weakest condition.

The parameters used in the models were found to be non-linearly related so non-linear regression analysis was used to develop the models. A number of assumptions were made to create a new set of independent variables required by the model development which are discussed below.

Pavement category, thickness of the asphalt layer and traffic density category were considered for model development. The Modified Structural Strength ratio was modelled as a function of time of collection of data after flooding (in days) for the AC pavement which has light traffic and a thickness of the asphalt layer of 45-60 mm. The data for the deterministic model were prepared following the criteria described below:

- The model is only applicable to lightly trafficked local roads with thin asphalt surfacing.
- The ratio of Modified Structural Strength of the pavement post-flood to Modified Structural Strength pre-flood (SNC_{ratio}) must be greater than 0.75 and less than or equal to 1.
- Post-flood reduction in deflection should be 25% to 40%.
- Time is less than 42 days, i.e. 6 weeks (t < 42 days), as the original dataset was collected within this time limit.

The model that was initially estimated for trial and error and run in SPSS is given in Equation (5.1). The Structural Strength ratio, SNC_{ratio}, is the dependent variable defining structural deterioration, while time is the independent variable. After performing a number of trial and error runs of non-linear regression analyses, Equation (5.2) was found to be the most appropriate model for the selected data set to describe the post-flood short term phase of flood affected pavement.

$$y = y_0 + a * EXP^{(-x/b)} \tag{5.1}$$

where,

y	=	SNC_{ratio}
x	=	t, time of collection of FWD data after flood, in days
a, b	=	coefficients
y_0	=	Constant

$$SNC_{ratio} = 1.032 - 0.034 \times EXP^{(t/21.5)} \tag{5.2}$$

SNC_{ratio}	=	Ratio of Modified Structural Strength of the pavement after time, t of flooding/ Modified Structural Strength before flooding, SNC_t/SNC_i
SNC_i	=	Modified Structural number before flooding
	=	$3.2 \times D_0(i)^{-0.63}$ (Paterson 1987)
$D_0(i)$	=	maximum deflection measured using a FWD before flooding
SNC_t	=	Modified structural strength or Modified structural number of the pavement after time, t of flooding
	=	$3.2 \times D_0(t)^{-0.63}$ (Paterson 1987)

$D_o(t)$	=	maximum deflection measured using a FWD after time, t of flooding
EXP	=	Exponential function
t	=	Time of collection of FWD data after flood in days (t < 42 days)

The model was developed with a coefficient of determination (R^2) of 0.785 when Sample size, N, was 34 with a 95% confidence interval. The model establishes good statistical correlation with less standard error (Tables 5.18 and 5.19).

Table 5.18: Parameter Estimates

Parameter	Estimate	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
y	1.032	.031	.969	1.095
a	.034	.023	-.012	.080
b	21.500	6.332	8.586	34.414

Table 5.19: Correlations of Parameter Estimates

	y	**a**	**b**
y	1.000	.975	.952
a	.975	1.000	.994
b	.952	.994	1.000

Equation (5.2) can be used to estimate the Modified Structural Number after flooding at each chainage if the initial, or before flooding, Modified Structural Number is known. SNC_{ratio} is plotted against time in Figure 5.53. It is a single parameter model which is dependent on the local conditions and can predict pavement behaviour within 42 days, i.e. up to six weeks after the flooding. Thus, the longer the duration of the flood, the greater the loss of strength.

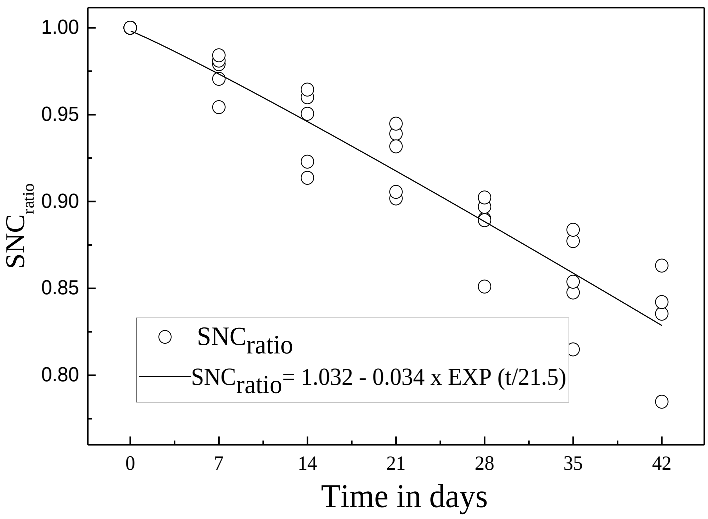


Figure 5.53: Decrease in structural strength within 42 days after flood

5.13 MODELLING STRUCTURAL DETERIORATION (SECOND MODEL)

A mechanistic-empirical, deterministic structural deterioration model was developed using the observational data of the local roads in the Brisbane City Council area affected by the January 2011 flood. The data were collected six weeks after the flood. This is the same data set as the first model discussed in Section 5.11. The FWD data of the lightly trafficked local street with thin AC pavements (with gravel base) were considered for the model as they were significantly affected during the flood. The data for the deterministic model were prepared following the criteria outlined below:

- Data for local roads with an asphalt layer thickness of 45-60 mm and gravel layer of 135-200 mm, traffic loading less than 1×10^5 (MESA < 0.1) over 20 years design life period were considered for the model.
- Data with a reduction in the post-flood deflection from 25% to 40% were used for the model.
- The ratio of the SNC of the pavement post-flood to the SNC of the pavement pre-flood must be greater than 0.75 and less than or equal to 1.
- Data with a greater than 30% decrease in the post-flood subgrade CBR were used for the model.
- Post-flood strength can be predicted within the time limit of 42 days, i.e. within 6 weeks of flood ($t < 42$ days), as the original dataset was collected within this time limit.

- The design traffic loading, Millions Equivalent Single Axles (MESA) is a constant for this model and the value was calculated over the normal design life using Equation (5.3) (Austroads 2012).

$$\mathrm{MESA} = 365 \times \mathrm{ADT} \times \%\mathrm{HV}/100 \times \mathrm{DF} \times \mathrm{LDF} \times \mathrm{N_{HVAG}} \times \mathrm{ESA/HVAG} \qquad (5.3)$$

where, ADT (Average Daily Traffic) is the average number of vehicles per day; %HV is the percentage of heavy vehicles, DF is the direction factor (proportion of ADT travelling in the same direction as the survey lane); LDF is the lane distribution factor (proportion of traffic in the same direction that is in the survey lane); $\mathrm{N_{HVAG}}$ is the average number of axle groups per heavy vehicle; ESA/HVAG is the average number of ESAs per heavy vehicle axle group (Austroads 2012). A sample dataset is shown in Figure 5.54 for loss of structural and subgrade strength with time.

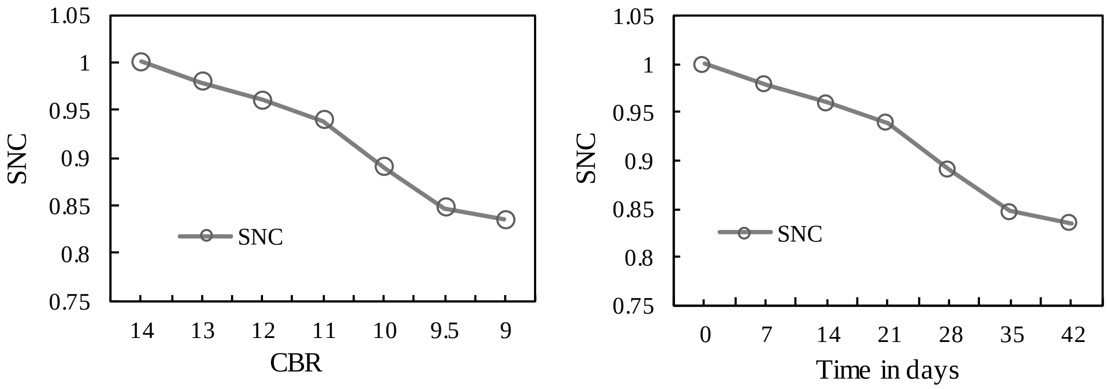


Figure 5.54: Deterioration curve for loss of structural and subgrade strength

The Modified Structural Number ratio for rapid deterioration phase of pavements after flooding was designated as $\mathrm{SNC_{rapid}}$. The model expresses the $\mathrm{SNC_{rapid}}$ as a function of time lapse in deflection measurement after flooding, subgrade CBR and MESA. The model was first estimated in the form of Equation (5.4). The $\mathrm{SNC_{rapid}}$, is the dependent variable defining the structural deterioration, while time, CBR and MESA are the independent variables. After performing a number of trial and error runs of the non-linear regression analyses, Equation (5.5) was found to be the most appropriate for the selected data set. Parameter estimates and correlations of parameter estimates are given in Tables 5.20 and 5.21.

$$\mathrm{SNC_{rapid}} = k_f\,[y_0 - a \times \mathrm{EXP}\left(b \times t - c \times (\mathrm{CBR} + \mathrm{MESA})\right)] \qquad (5.4)$$

$$\mathrm{SNC_{rapid}} = k_f\,[1.227 - 0.312\mathrm{EXP}\left(0.011t - 0.024\,(\mathrm{CBR} + \mathrm{MESA})\right)] \qquad (5.5)$$

where,

SNC_{rapid}	=	Ratio of the post-flood SNC value (at time t) divided by the SNC value pre-flood, SNC_t/SNC_i
y_0	=	Constant
b, c	=	Coefficients
SNC_i	=	Modified Structural number before flooding
	=	$3.2 \times D_0(i)^{-0.63}$ (Paterson 1987)
$D_o(i)$	=	maximum deflection measured using a FWD before flooding
SNC_t	=	Modified structural strength or Modified structural number of the pavement after time, t of flooding
	=	$3.2 \times D_0(t)^{-0.63}$ (Paterson 1987)
$D_o(t)$	=	maximum deflection measured using a FWD after time, t of flooding
EXP	=	Exponential function
t	=	Time of collection of FWD data after flood in days (t < 42 days)
CBR	=	Subgrade strength
MESA	=	Millions of equivalent standard axles over 20-year design life
k_f	=	local calibration factor for different types of pavement (default=1)

Table 5.20: Parameter Estimates

Parameter	Estimate	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
y	1.227	.148	.926	1.528
a	.312	.151	.004	.620
b	.011	.005	.000	.021
c	.024	.012	-.001	.050

Table 5.21: Correlations of Parameter Estimates

	y	a	b	c
y	1.000	.994	-.989	-.938
a	.994	1.000	-.996	-.895
b	-.989	-.996	1.000	.897
c	-.938	-.895	.897	1.000

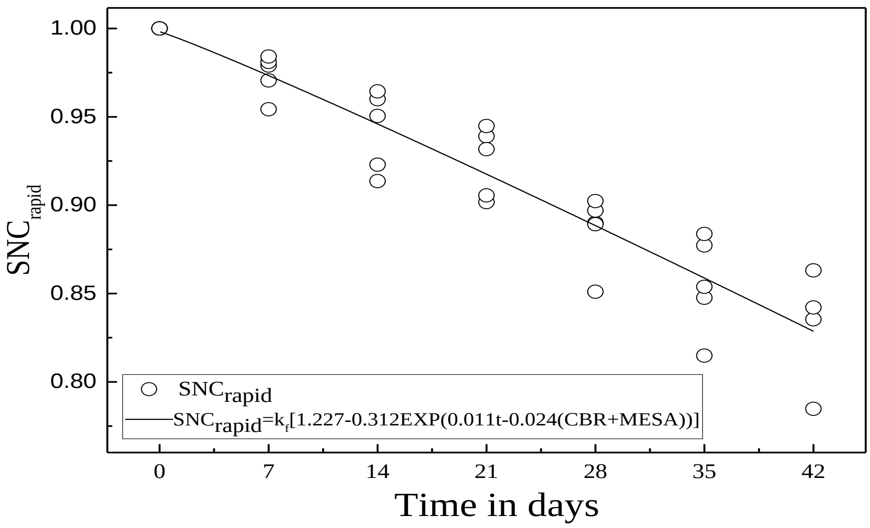


Figure 5.55: Rapid deterioration of structural strength within 42 days of flood

The model was developed with a coefficient of determination (R^2) of 0.947, sample size (N) was 34 with a 95% confidence interval. The regression coefficients of the independent variables in Equation (5.5) are significantly correlated with the least standard error. A comparison of the actual data set with the predicted values of SNC_{rapid} from the model was significant.

The three parameter model is dependent on the local conditions and pavement types. It can predict the deterioration of pavements within 42 days, i.e. within six weeks of flooding. Equation (5.5) can be used to estimate the post-flood SNC if the pre-flood SNC is known. Figure 5.55 shows that the modified structural strength ratio, represented by the SNC_{rapid} reduces after flood. After the time limit of six weeks, this model becomes invalid assuming that the pavement starts a 'dry back period'.

Tables 5.22 to 5.24 compare the mean of SNC_{rapid} actual and predicted values using paired sample t-tests.

Table 5.22: General Statistics

Pair	**Mean**	**N**	**Std. Deviation**	**Std. Error Mean**
SNC_{rapid}	.918674	34	.0601215	.0103108
Predicted Values	.918672	34	.0584860	.0100303

Table 5.23: T test- Paired Samples Correlations

Pair	N	Correlation	p-value
SNC_{rapid} & Predicted Values	34	.973	.000

Table 5.24: Paired Samples Test between SNC_{rapid} - Predicted Values

Paired Differences					**t**	**Degrees of freedom**	**p-value**
Mean	**STD**	**Std. Error Mean**	**95% Confidence Interval of the Difference**				
			Lower	**Upper**			
.0000011	.013904	.002385	-.004850	.004852	.000	33	1.000

After a certain period of time, the pavement becomes fully saturated and remains in this saturated state until the flood waters recede and the pavement dries out during the dry weather period. The proposed model would enable pavement engineers to quantify the post-flood rapid deterioration of the structural strength. The model is sufficiently robust and can be adapted to other regions in Australia by calibrating for local conditions.

Although, the model could help the decision making process with the road closures, it may not always be practical to close the roads for a long time due to the impact on the road users. In that case, following a flood, the local road agency can consider reducing the axle load limit.

5.14 SUMMARY

The models presented in this chapter can predict the deterioration of structural strength after floods. But the decision of road opening relies more on socio-economic and geotechnical factors, like slope instability and safety issues. This model can help road asset engineers make informed decisions on structural deterioration of flood affected roads, how long it can take to recover from structure and subgrade damage, and returning the subgrade from post-flood condition to pre-flood conditions.

6. ASSESSMENT AND MODELLING OF SURFACE CONDITION OF FLOOD AFFECTED PAVEMENTS

6.1 ANALYSIS OF SURFACE CONDITION OF FLOOD AFFECTED QUEENSLAND ROADS

Following an unprecedented number of natural disasters between 2010 and 2013, many Queensland communities, as well as key road, rail, port and waterway infrastructure, suffered extensive damage. As a consequence, TMR reconstructed large sections of the state-controlled road network through the Transport Network Reconstruction Program (TNRP) (TMR 2015). These reconstruction works, costing approximately AU$6.4 billion, were completed on approximately 8,741 km of the state-controlled road network, approximately 1,733 structures (including bridges and culverts), approximately 1,421 locations requiring earthworks and batters, and approximately 3,335 locations needing silt and debris cleared. Between November 2010 and March 2011, approximately 9,170 km of Queensland's state-owned road network was damaged, most of which was later recovered through TNRP (TMR 2015).

In this chapter, an assessment and analysis of the surface condition of the flood affected pavements of TMR, Queensland was conducted. After the assessment of the flood affected pavements, models for deterioration of surface conditions, such as rutting and roughness, were developed. It should be noted for the clarity of analysis that in this chapter, flood-affected pavements (roads) refers to the roads that needed rehabilitation as a result of the damage caused by flood water. Thus, this study used the surface condition data of the flood-affected pavement sections that were rehabilitated after flooding by TMR, Queensland.

Initially, all pavement segments with pre- and post-flood data were screened for analysis. The study verified a number of things while selecting pavement sections for analysis and preparing the database to categorise pavement sections based on their pre-flood rutting values which are discussed as follows:

- The common lane information of every pavement section included in the analysis was lane number 1, through traffic lane and inner wheel path.
- This study considered gradual rise of floodwater on pavements or flooding caused by heavy rainfall events in the area, no flash flooding event was considered as damage caused by flash flooding can be catastrophic.
- In this chapter, the term 'flood-affected pavements' refers to the pavement sections rehabilitated after flooding. Depth of flood water on the pavement during the flooding event was not considered. However, average rainfall intensity during the month of flooding was checked.
- To assess the impact of flood on pavements, road sections with pre- and post-flood rutting and roughness data were considered for the analysis.
- Date of flooding, date of rutting and roughness data collection, and rehabilitation date were recorded and considered in the analysis.
- It was observed that post-flood rehabilitation tends to improve the road condition and such sections had a decrease in rutting and roughness values. Hence, pavement sections with post-flood data collected after post-flood rehabilitation were not considered for the analysis as there was evidence of improvement in pavement condition.
- It should be noted that during the analysis of individual roads and modelling, and preparing the database for categorizing the pavement sections based on pre-flood rutting values, pavement sections which had post-flood rutting values greater than pre-flood rutting were considered. Pavement sections with pre-flood rutting values greater than post-flood rutting values were discarded to avoid confusion.

SPSS (SPSS Version 22.0 2013) was used to perform the statistical analysis. It should be noted that rutting values are expressed in mm and roughness values are expressed in IRI throughout this study. Depending on the pavement material, traffic density, flood intensity, and flooding frequency, roads perform differently. These factors were taken into consideration while conducting the analysis. To define the possible distress limits for deflection, roughness and rutting for service life of pavement, Table 6.1 was considered as a general guideline (Austroads 2003).

Table 6.1: Definition of possible distress limits for deflection, roughness and rutting for service life of pavement (Austroads 2003)

Road Function	**Surface Deflection (D_0, mm)**	**Roughness Limit (IRI, m/Km)**	**% Road Length with Rut Depth > 20 mm**
Freeways, etc.	0.8	4.2	10
Highways and main roads (100 km/h)	0.85	4.2	10
Highways and main roads (80 km/h)	0.9	5.4	20
Other sealed local roads	1.6	No defined limit	No defined limit

The following two guidelines (Equations (6.1) and (6.2)) were used to calculate the rate of increase or decrease in rutting and roughness values every year. A positive sign indicates an increase, and a negative sign indicates a decrease, in the rate of rutting or roughness.

$$\text{Rate of Rutting (mm/year)} = \frac{\text{Rutting at Year 2} - \text{Rutting at Year 1}}{\text{Year 2} - \text{Year 1}} \tag{6.1}$$

$$\text{Rate of Roughness (IRI/year)} = \frac{\text{Roughness at Year 2} - \text{Roughness at Year 1}}{\text{Year 2} - \text{Year 1}} \tag{6.2}$$

6.2 AVERAGE MONTHLY RAINFALL DATA

Average monthly rainfall of the areas with the specific road locations were obtained from the Australian Bureau of Meteorology website to accurately determine the flooding time (Tables 6.2 to 6.4). Some of the roads analysed in this chapter were flooded several times in the last six years.

Table 6.2: Average Monthly Rainfall (mm) for Rockhampton area (BoM 2016b)

Year	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
2009	68.4	196.4	24	48.6	10.2	7.8	0.2	0.2	0	11.2	21.4	195.4	583.8
2010	62.8	256.2	142.6	4.2	18.6	16.4	16.2	65	147	50.6	120.6	523.8	1424
2011	114.4	65	315.4	41.8	19.4	23.4	9.2	94.4	0.4	61	5.2	152	901.6
2012	120.8	155	123.8	13.2	38.4	65.8	115	14	31.2	41	40.4	7.4	766
2013	555.6	109.6	207.4	99.2	121.4	7	18.2	0.6	5.8	26.2	59.8	2.8	1213.6
2014	178	225.2	247.8	69.2	11.6	6.2	0.8	34	85.6	2.8	13	154.2	1028.4
2015	150.2	281.8	3.4	49	19.2	40.4	13.4	10.8	1.6	8.8	60.4	39.2	678.2
2016	34	252.8	175	2.2	4.8	111.8	254.4						

Note: Station: Rockhampton Aero, Station Number: 039083, State: QLD, Opened: 1939, Status: Open, Latitude: 23.38°S · Longitude: 150.48°E · Elevation: 10 m

Table 6.3: Average Monthly Rainfall (mm) for Yeppoon area (BoM 2016c)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
2009	193.6	278	50.8	113.8	38.4	23.8	0.6	1.8	4	5	44.4	120.4	874.6
2010	94.2	510	113.2	43.8	26.4	28.4	25.2	74.6	156	52.2	267	459.8	1850.8
2011	140.6	136	330	159	73.4	50.8	9.2	72	1.4	28.6	7.6	138	1146.6
2012	121.4	146.2	270.2	15	69.8	81.2	123.4	7	4.2	82.8	55.6	11	987.8
2013	NA	280.8	314.4	130.8	152.	15.8	30.6	1.2	1.8	2.2	91.6	27.8	
2014	134.2	144.6	615.4	133	86.8	8	6.8	29.8	101	1	9	171.2	1440.8
2015	311.8	347.6	57.8	80.8	19.4	37	28.6	13.8	8.8	20.6	54.2	37	1017.4
2016	32.4	146.8	143.4	51.6	2.8	141.6	384.2						
Note: Station: Yeppoon The Esplanade, Station Number: 033294, State: QLD, Opened: 1993, Status: Open, Latitude: 23.14°S · Longitude: 150.75°E · Elevation: 6 m													

Table 6.4: Average Monthly Rainfall (mm) near Ipswich area (BoM 2016c)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
2009	66	84	31	185	254	68	2	5	30	30	112	82	949
2010	19	220	141	44	28	5	44	63	92	148	30	350	1184
2011	252	123	108	56	63	8	13	44	14	105	21	138	946
2012	168	76	59	72	8	78	42	2	7	18	94	24	648
2013	229	171	91	88	34	58	14	5	20	9	94	NA	
2014	NA	4	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	
2015	NA	NA	NA	NA	NA	NA	NA	NA	NA	0	164	97	
2016	55	87	47	4	11	209	25						
Station Name: One Mile Bridge Alert (nearest station in the Ipswich area where rainfall data for the year 2009 to 2012 was available), Station Number: 040836 · State: QLD · Opened: 1990 · Status: Open · Latitude: 27.63°S · Longitude: 152.75°E · Elevation: 0 m													

6.3 ANALYSIS OF REHABILITATED FLOOD-AFFECTED ROADS

Initially, this study checked data from approximately 58,000 flood affected pavement sections (each 100 m in length) of TMR, Queensland. After checking data, 21,450 pavement sections were identified as having both pre- and post-flood rutting and roughness data. These sections included any samples with pre- and post-flood data without considering the effect of immediate repair or rehabilitation. Mean and standard deviation of rutting and roughness values of the 21,450 samples between the year 2009 and 2015 are shown in Table 6.5. The term pre-flood indicates that these pavement sections were first flooded in either 2010 or 2011 and data were collected before the flood (in 2009 or 2010). Hence, for the network level analysis, the date is tentative. However, for individual roads and sections, accurate date of data collection are also included as necessary.

Mean pre-flood rutting of 21,450 samples is greater than mean post-flood rutting. Mean pre-flood roughness is slightly lower than mean post-flood roughness (Table 6.5). These samples were from different flood affected pavements across Queensland. Rehabilitation and repair of these pavements were conducted at different times from 2010 to 2013 as a part of the Transport Network Reconstruction Program. Therefore, overall average rutting and roughness values gradually improved in 2012, 2013, 2014 and 2015.

Table 6.5: Mean and standard deviation of rutting and roughness (criteria of sample selection: having pre- and post-flood rutting and roughness data)

Variables	Year	Sample Size (N)	Mean	Std. Deviation
Rutting (mm)	Pre-flood	21,450	7.354	5.163
	Post-flood	21,450	6.554	3.921
	2012	20,083	6.474	4.049
	2013	18,939	5.864	3.560
	2014	19,041	5.373	3.069
	2015	14,532	4.905	2.230
IRI (m/Km)	Pre-flood	21,449	2.720	1.083
	Post-flood	21,433	2.723	1.002
	2012	20,059	2.637	0.974
	2013	19,004	2.466	0.986
	2014	19,050	2.205	0.974
	2015	14,532	1.946	0.740

The data were screened again and sections with post-flood rutting values greater than pre-flood rutting values were selected resulting in 9000 road sections. Table 6.6 presents the mean and standard deviation of these 9000 road segments. Mean pre-flood rutting and roughness are lower than mean post-flood rutting and roughness values. There were decreases in mean rutting and roughness from 2012, 2013, 2014 and 2015 due to rehabilitation works.

Table 6.6: Mean and standard deviation of rutting and roughness (criteria of sample selection: pre-flood rutting < post-flood rutting)

Variables	Year	Sample Size (N)	Mean	Std. Deviation
Rutting (mm)	Pre-flood	9,000	5.289	3.191
	Post-flood	9,000	7.277	4.308
	2012	8,550	6.273	4.227
	2013	8,121	5.810	3.762
	2014	8,257	5.420	2.911
	2015	6,198	4.934	2.181
IRI (m/Km)	Pre-flood	9,000	2.646	1.077
	Post-flood	8,995	2.696	0.989
	2012	8,476	2.586	0.964
	2013	8,040	2.432	0.980
	2014	8,223	2.218	1.004
	2015	6,216	1.948	0.729

To determine if flooding had significant impacts on rutting and roughness, paired sample t-tests were used. The mean of ten pairs of samples collected from the same locations were compared. Ten pairs of samples are described as follows:

1. Pre- and post-flood rutting
2. Post-flood rutting and rutting in 2012
3. Rutting in 2012 and 2013
4. Rutting in 2013 and 2014
5. Rutting in 2014 and 2015
6. Pre- and post-flood roughness
7. Post-flood roughness and roughness in 2012

8. Roughness in 2012 and 2013
9. Roughness in 2013 and 2014 and
10. Roughness in 2014 and 2015.

The results of the paired sample t-tests are shown in Tables 6.7 and 6.8. Table 6.7 indicates that each of these ten pairs of samples was significantly correlated with lower standard error. Statistical comparison of the mean of the ten pairs of samples are explained as follows:

1. Mean post-flood rutting value was significantly higher than mean pre-flood rutting value ($p < 0.05$).
2. Mean post-flood rutting value was significantly higher than mean rutting value in 2012 ($p < 0.05$).
3. Mean rutting value in 2012 was significantly higher than mean rutting value in 2013 ($p < 0.05$).
4. Mean rutting value in 2013 was significantly higher than mean rutting value in 2014 ($p < 0.05$).
5. Mean rutting value in 2014 was significantly higher than mean rutting value in 2015 ($p < 0.05$).
6. Mean post-flood roughness value was significantly higher than mean pre-flood roughness value ($p < 0.05$).
7. Mean post-flood roughness value was significantly higher than mean roughness value in 2012 ($p < 0.05$).
8. Mean roughness value in 2012 was significantly higher than mean roughness value in 2013 ($p < 0.05$).
9. Mean roughness value in 2013 was significantly higher than mean roughness value in 2014 ($p < 0.05$).
10. Mean roughness value in 2014 was significantly higher than mean roughness value in 2015 ($p < 0.05$).

Table 6.7: Paired sample summaries of ten pairs of samples (rutting and roughness)

Pair	Variables	Mean	Sample Size (N)	Std. Deviation	Std. Error Mean	Correlation Between pairs
Pair 1	Rut (Pre-flood)	5.289	9,000	3.191	0.034	0.847
	Rut (Post-flood)	7.277		4.308	0.045	
Pair 2	Rut (Post-flood)	7.257	8,550	4.320	0.047	0.585
	Rut 2012	6.273		4.227	0.046	
Pair 3	Rut 2012	6.236	7,976	4.203	0.047	0.596
	Rut 2013	5.815		3.788	0.042	
Pair 4	Rut 2013	5.770	7,705	3.799	0.043	0.475
	Rut 2014	5.365		2.762	0.031	
Pair 5	Rut 2014	5.380	5,751	2.945	0.039	0.503
	Rut 2015	4.931		2.158	0.028	
Pair 6	IRI (Pre-flood)	2.646	8,995	1.077	0.011	0.792
	IRI (Post-flood)	2.696		0.989	0.010	
Pair 7	IRI (Post-flood)	2.699	8,471	0.992	0.011	0.672
	IRI 2012	2.586		0.963	0.010	
Pair 8	IRI 2012	2.595	7,828	0.969	0.011	0.705
	IRI 2013	2.442		0.981	0.011	
Pair 9	IRI 2013	2.397	7,588	0.953	0.011	0.557
	IRI 2014	2.215		1.002	0.012	
Pair 10	IRI 2014	2.225	5,730	1.075	0.014	0.495
	IRI 2015	1.954		0.729	0.010	

Table 6.8: Paired sample t-test for ten pairs of samples (rutting and roughness)

Pair	Variables	Paired Differences					t	degrees of freedom	p-value
		Mean	Std. Dev.	Std. Error Mean	95% Confidence Interval of the Difference				
					Lower	Upper			
Pair 1	Rut (Pre-flood) & Rut (Post-flood)	-1.988	2.334	0.025	-2.036	-1.940	-80.820	8,999	0.00
Pair 2	Rut (Post-flood) & Rut 2012	0.984	3.897	0.042	0.901	1.067	23.351	8,549	0.00
Pair 3	Rut 2012 & Rut 2013	0.421	3.609	0.040	0.342	0.500	10.420	7,975	0.00
Pair 4	Rut 2013 & Rut 2014	0.404	3.476	0.040	0.327	0.482	10.210	7,704	0.00
Pair 5	Rut 2014 & Rut 2015	0.449	2.635	0.035	0.381	0.517	12.921	5,750	0.00
Pair 6	IRI (Pre-flood) & IRI (Post-flood)	-0.050	0.671	0.007	-0.064	-0.036	-7.049	8,994	0.00
Pair 7	IRI (Post-flood) & IRI 2012	0.113	0.792	0.009	0.097	0.130	13.182	8,470	0.00
Pair 8	IRI 2012 & IRI 2013	0.154	0.748	0.008	0.137	0.170	18.151	7,827	0.00
Pair 9	IRI 2013 & IRI 2014	0.183	0.921	0.011	0.162	0.204	17.297	7,587	0.00
Pair 10	IRI 2014 & IRI 2015	0.271	0.955	0.013	0.247	0.296	21.515	5,729	0.00

In summary, statistically increases in rutting and roughness values after flood were found for Queensland roads as shown by pairs 1 and 6 and pairs 2 and 7. However, pairs 3 and 8, 4 and 9 and 5 and 10 show a decrease in rutting and roughness over an interval of one year which was mainly due to the rehabilitation of different roads from 2010 to 2015. These results indicate that flooding had a significant impact on rapid increases in rutting and roughness of Queensland roads.

6.4 PRE-FLOOD RUTTING AS A CONTROLLING FACTOR OF POST-FLOOD RUTTING

After comparing the frequency of data, distribution of data and range of highest and lowest pre- and post-flood rutting and roughness values, pre-flood rutting was identified as one of the main factors contributing to and controlling the increase in post-flood rutting. In the 21,450 flood affected pavement sections, it was generally observed that the lower the pre-flood rutting value, the lower the post-flood rutting value, if there was no repair or rehabilitation work carried out immediately after the flooding. In the database

(21,450 samples), 90% of samples with pre-flood rutting lower than post-flood rutting had pre-flood rutting values up to 9.4 mm and post-flood rutting values up to 12.6 mm and the remaining 10% of samples had pre-flood rutting varying from 9.5-28.2 mm and post-flood rutting varying from 12.7-44.7 mm.

Differences in pre- and post-flood rutting was designated as $\Delta Rut_{post\text{-}flood}$ and differences in pre- and post-flood roughness was designated as $\Delta IRI_{post\text{-}flood}$. $\Delta Rut_{post\text{-}flood}$ and $\Delta IRI_{post\text{-}flood}$ were calculated for each sample by subtracting post-flood rutting from pre-flood rutting ($\Delta Rut_{post\text{-}flood} = Rut_{post\text{-}flood} - Rut_{pre\text{-}flood}$) and post-flood roughness from pre-flood roughness ($\Delta IRI_{post\text{-}flood} = Roughness_{post\text{-}flood} - Roughness_{pre\text{-}flood}$). $\Delta Rut_{post\text{-}flood}$ was less than 4.4 mm for the 90% of samples and varied from 4.5-40.2 mm in the other 10%. The 90% samples with pre-flood roughness lower than post-flood roughness, had pre-flood IRI values up to 3.63 and post-flood IRI values up to 4.13. The other 10% had pre-flood IRI varying from 3.63-7.63 and post-flood IRI varying from 4.14-14.68. $\Delta IRI_{post\text{-}flood}$ for the 90% of samples was less than 0.74 and varied from 0.75-12.19 in the other 10%.

The 9,000 samples with pre-flood rutting lower than post-flood rutting were used to categorize pavement sections into six groups based on their pre-flood rutting values which are as follows.

- Group one: Pre-flood Rutting is less than 4 mm
- Group two: Pre-flood Rutting is within the range of 4.1 mm to 8 mm
- Group three: Pre-flood Rutting is within the range of 8.1 mm to 12 mm
- Group four: Pre-flood Rutting is within the range of 12.1 mm to 16 mm
- Group five: Pre-flood Rutting is within the range of 16.1 mm to 20 mm
- Group six: Pre-flood Rutting is greater than 20 mm.

The frequency of data, distribution of data and range of highest and lowest pre- and post-flood rutting values were considered for the grouping.

Independent samples t-tests were used to compare the mean $\Delta Rut_{post\text{-}flood}$ among the six groups of samples (Tables 6.9 and 6.10). The results are discussed as follows:

- Mean $\Delta Rut_{post\text{-}flood}$ for Group 1 is significantly lower than the mean $\Delta Rut_{post\text{-}flood}$ for Group 2 ($p < 0.05$).
- Mean $\Delta Rut_{post\text{-}flood}$ for Group 2 is significantly lower than the mean $\Delta Rut_{post\text{-}flood}$ for Group 3 ($p < 0.05$).

- Mean $\Delta Rut_{post\text{-}flood}$ for Group 3 is significantly lower than the mean $\Delta Rut_{post\text{-}flood}$ for Group 4 ($p < 0.05$).

Table 6.9: Group statistics for independent samples t-test to compare mean $\Delta Rut_{post\text{-}flood}$ among the six groups

Comparison	Name of Group	Sample Size (N)	Mean $\Delta Rut_{post\text{-}flood}$	Std. Deviation	Std. Error Mean
Group 1 & 2	Pre-flood Rut <4 mm	3,845	1.681	1.692	0.027
	Pre-flood Rut: 4.1 mm to 8mm	3,801	1.939	2.218	0.036
Group 2 & 3	Pre-flood Rut: 4.1 mm to 8mm	3,801	1.939	2.218	0.036
	Pre-flood Rut: 8.1 mm to 12mm	961	2.762	3.392	0.109
Group 3 & 4	Pre-flood Rut: 8.1 mm to 12mm	961	2.762	3.392	0.109
	Pre-flood Rut: 12.1 mm to 16mm	253	3.548	3.978	0.250
Group 4 & 5	Pre-flood Rut: 12.1 mm to 16mm	253	3.548	3.978	0.250
	Pre-flood Rut: 16.1 mm to 20 mm	89	3.813	4.084	0.433
Group 5 & 6	Pre-flood Rut: 16.1 mm to 20 mm	89	3.813	4.084	0.433
	Pre-flood Rut > 20 mm	33	4.282	4.676	0.814

Although mean $\Delta Rut_{post\text{-}flood}$ for Group 4 was lower than mean $\Delta Rut_{post\text{-}flood}$ for Group 5, the difference was not statistically significantly ($p > 0.05$). Mean $\Delta Rut_{post\text{-}flood}$ for Group 5 was lower than mean $\Delta Rut_{post\text{-}flood}$ for Group 6, but the difference was also not statistically significant ($p > 0.05$). The lack of a significant result for these two comparisons could be due to the reduced number of samples in these groups as there were fewer samples with the criterion of pre- and post-flood rutting >16 mm. There were only 89 samples for Group 5 (Pre-flood Rutting 16.1 mm to 20 mm) and 33 samples for Group 6 (Pre-flood Rutting greater than 20 mm). Although there were samples with pre-flood rutting >16 mm in the rest of the 21,450 samples, the post-flood data collection was probably conducted after rehabilitation (as data showed a significant improvement in road condition post-flood). This was probably because most of the road sections with higher pre-flood rutting needed to be rehabilitated/repaired immediately after the flood and before the next survey due to the poor condition of the road.

Table 6.10: Independent samples t-test comparing mean $\Delta Rut_{post\text{-}flood}$

Equal variances assumed/ Equal variances not assumed	Levene's Test for Equality of Variances		t-test for Equality of Means						
	F	Sig.	t	degrees of freedom	p-value	Mean Difference	Std. Error Difference	95% Confidence Interval of the Difference	
								Lower	Upper
Group 1 and 2: Pre-flood rutting less than 4 mm and 4.1 mm to 8 mm									
Equal variances assumed	109.662	.000	-5.720	7644	.000	-0.258	0.045	-0.346	-0.169
Equal variances not assumed			-5.711	7105.5	.000	-0.258	0.045	-0.346	-0.169
Group 2 and 3: Pre-flood rutting 4.1 mm to 8 mm and 8.1 mm to 12 mm									
Equal variances assumed	150.469	.000	-9.121	4760	.000	-.823	.090	-1.000	-.646
Equal variances not assumed			-7.146	1175.186	.000	-.823	.115	-1.049	-.597
Group 3 and 4: Pre-flood rutting 8.1 mm to 12 mm and 12.1 mm to 16 mm									
Equal variances assumed	13.596	.000	-3.158	1212	.002	-.786	.249	-1.274	-.298
Equal variances not assumed			-2.879	354.305	.004	-.786	.273	-1.323	-.249
Group 4 and 5: Pre-flood rutting 12.1 mm to 16 mm and 16.1 mm to 20 mm									
Equal variances assumed	.010	.922	-.538	340	.591	-.266	.494	-1.237	.705
Equal variances not assumed			-.531	150.705	.596	-.266	.500	-1.253	.722
Group 5 and 6: Pre-flood rutting 16.1 mm to 20 mm and greater than 20 mm									
Equal variances assumed	.158	.692	-.541	120	.590	-.468	.866	-2.183	1.247
Equal variances not assumed			-.508	51.168	.614	-.468	.922	-2.319	1.382

The independent samples t-tests were used to compare the mean pre- and post-flood IRI and $\Delta IRI_{post\text{-}flood}$ for the six groups of pavements (Tables 6.11 and 6.12). Mean pre- and post-flood IRI of Group 1 were significantly lower than mean pre- and post-flood IRI of Group 2 ($p < 0.05$). Mean $\Delta IRI_{post\text{-}flood}$ of Group 1 was greater than mean $\Delta IRI_{post\text{-}flood}$ of Group 2.

Mean pre- and post-flood IRI of Group 2 were significantly lower than mean pre- and post-flood IRI of Group 3 ($p < 0.05$). Mean pre-flood IRI of Group 3 were lower than mean pre- flood IRI of Group 4 but statistically not significant ($p > 0.05$). Mean post-flood IRI of Group 3 was significantly lower than mean post-flood IRI of Group 4 ($p < 0.05$). Mean $\Delta IRI_{post\text{-}flood}$ of Group 3 was lower than mean $\Delta IRI_{post\text{-}flood}$ of Group 4 but statistically not significant ($p > 0 .05$).

Mean pre- and post-flood IRI of Group 4 were lower than mean pre- and post-flood IRI of Group 5 but statistically not significant ($p > 0.05$). Mean pre- and post-flood IRI and $\Delta IRI_{post\text{-}flood}$ of Group 5 were significantly lower than pre- and post-flood mean IRI and $\Delta IRI_{post\text{-}flood}$ of Group 6 ($p < 0.05$).

Hence, it can be concluded that pavements with lower pre-flood rutting are highly likely to have lower pre- and post-flood roughness. The higher the pre-flood rutting, the higher the chance of higher pre- and post-flood roughness.

Table 6.11: Independent sample t-test of pre- and post-flood roughness and differences in pre- and post-flood roughness

Rutting Criteria	Variables	Group Name	Sample Size (N)	Mean	Std. Deviation	Std. Error Mean
Group 1 and 2	IRI (Pre-flood)	Pre-flood Rut <4 mm	3,845	2.374	0.891	0.014
		Pre-flood Rut: 4.1 mm to 8 mm	3,801	2.785	1.101	0.018
	IRI (Post-flood)	Pre-flood Rut <4 mm	3,841	2.428	0.835	0.013
		Pre-flood Rut: 4.1 mm to 8mm	3,800	2.835	0.984	0.016
	$\Delta IRI_{post-flood}$	Pre-flood Rut <4 mm	3,845	0.053	0.554	0.009
		Pre-flood Rut: 4.1 mm to 8 mm	3,801	0.051	0.697	0.011
Group 2 and 3	IRI (Pre-flood)	Pre-flood Rut: 4.1 mm to 8 mm	3,801	2.785	1.101	0.018
		Pre-flood Rut: 8.1 mm to 12 mm	961	2.966	1.212	0.039
	IRI (Post-flood)	Pre-flood Rut: 4.1 mm to 8 mm	3,800	2.835	0.984	0.016
		Pre-flood Rut: 8.1 mm to 12 mm	961	2.977	1.017	0.033
	$\Delta IRI_{post-flood}$	Pre-flood Rut: 4.1 mm to 8 mm	3,801	0.051	0.697	0.011
		Pre-flood Rut: 8.1 mm to 12 mm	961	0.015	0.742	0.024
Group 3 and 4	IRI (Pre-flood)	Pre-flood Rut: 8.1 mm to 12 mm	961	2.966	1.212	0.039
		Pre-flood Rut: 12.1 mm to 16 mm	253	3.073	1.504	0.095
	IRI (Post-flood)	Pre-flood Rut: 8.1 mm to 12 mm	961	2.977	1.017	0.033
		Pre-flood Rut: 12.1 mm to 16 mm	253	3.135	1.168	0.073
	$\Delta IRI_{post-flood}$	Pre-flood Rut: 8.1 mm to 12 mm	961	.0151	0.742	.02394
		Pre-flood Rut: 12.1 mm to 16 mm	253	.0617	0.939	.05903
Group 4 and 5	IRI (Pre-flood)	Pre-flood Rut: 12.1 mm to 16 mm	253	3.073	1.504	0.095
		Pre-flood Rut: 16.1 mm to 20 mm	89	3.331	1.100	0.117
	IRI (Post-flood)	Pre-flood Rut: 12.1 mm to 16 mm	253	3.135	1.168	0.073
		Pre-flood Rut: 16.1 mm to 20 mm	89	3.360	1.191	0.126
	$\Delta IRI_{post-flood}$	Pre-flood Rut: 12.1 mm to 16 mm	253	0.062	0.939	0.059
		Pre-flood Rut: 16.1 mm to 20 mm	89	0.029	0.699	0.074
Group 5 and 6	IRI (Pre-flood)	Pre-flood Rut: 16.1 mm to 20 mm	89	3.331	1.100	0.117
		Pre-flood Rut > 20 mm	33	3.910	1.529	0.266
	IRI (Post-flood)	Pre-flood Rut: 16.1 mm to 20 mm	89	3.360	1.191	0.126
		Pre-flood Rut > 20 mm	33	4.466	3.331	0.580
	$\Delta IRI_{post-flood}$	Pre-flood Rut: 16.1 mm to 20 mm	89	0.029	0.699	0.074
		Pre-flood Rut > 20 mm	33	0.556	2.681	0.467

Table 6.12: Group statistics for independent sample t-tests of pre- and post-flood roughness

Rutting Criteria	Variable	Equal variances assumed/ Equal variances not assumed	Levene's Test for Equality of Variances		t-test for Equality of Means						
			F	Sig.	t	Degrees of freedom	p-value	Mean Difference	Std. Error Difference	95% Confidence Interval of the Difference	
										Lower	Upper
Group 1 and 2	IRI (Pre-flood)	Equal variances assumed	72.34	.000	-17.955	7,644	.000	-.411	.023	-.456	-.366
		Equal variances not assumed			-17.933	7,293	.000	-.411	.023	-.456	-.366
	IRI (Post-flood)	Equal variances assumed	75.32	.000	-19.489	7,639	.000	-.407	.021	-.448	-.366
		Equal variances not assumed			-19.472	7,416	.000	-.407	.021	-.448	-.366
	$\Delta IRI_{post\text{-}flood}$	Equal variances assumed	17.83	.000	.171	7,644	.864	.002	.014	-.026	.031
		Equal variances not assumed			.170	7,239	.865	.002	.014	-.026	.031
Group 2 and 3	IRI (Pre-flood)	Equal variances assumed	7.584	.006	-4.443	4,760	.000	-.180	.041	-.260	-.101
		Equal variances not assumed			-4.196	1,387	.000	-.180	.043	-.265	-.096
	IRI (Post-flood)	Equal variances assumed	7.193	.007	-3.977	4,759	.000	-.142	.036	-.212	-.072
		Equal variances not assumed			-3.900	1,448	.000	-.142	.036	-.214	-.071
	$\Delta IRI_{post\text{-}flood}$	Equal variances assumed	.339	.560	1.404	4,760	.160	.036	.026	-.014	.086
		Equal variances not assumed			1.353	1,418	.176	.036	.026	-.016	.088
Group 3 and 4	IRI (Pre-flood)	Equal variances assumed	1.054	.305	-1.195	1,212	.232	-.108	.090	-.285	.069
		Equal variances not assumed			-1.055	342.958	.292	-.108	.102	-.309	.093
	IRI (Post-flood)	Equal variances assumed	2.364	.124	-2.128	1,212	.034	-.158	.074	-.304	-.012
		Equal variances not assumed			-1.964	359	.050	-.158	.080	-.316	.000
	$\Delta IRI_{post\text{-}flood}$	Equal variances assumed	.017	.895	-.838	1,212	.402	-.047	.056	-.156	.063
		Equal variances not assumed			-.732	339	.465	-.047	.064	-.172	.079

Table 6.13: Group statistics for independent sample t-tests of pre- and post-flood roughness

Rutting Criteria	Variable	Equal variances assumed/ Equal variances not assumed	Levene's Test for Equality of Variances		t-test for Equality of Means Criteria						
			F	Sig.	t	Degrees of freedom	p-value	Mean Difference	Std. Error Difference	95% Confidence Interval of the Difference: Lower	95% Confidence Interval of the Difference: Upper
Group 4 and 5	IRI (Pre-flood)	Equal variances assumed	.151	.698	-1.484	340	.139	-.258	.174	-.600	.084
		Equal variances not assumed			-1.719	210	.087	-.258	.150	-.554	.038
	IRI (Post-flood)	Equal variances assumed	.673	.413	-1.556	340	.121	-.225	.145	-.510	.059
		Equal variances not assumed			-1.542	152	.125	-.225	.146	-.514	.063
	$\Delta IRI_{post\text{-}flood}$	Equal variances assumed	.367	.545	.301	340	.764	.033	.109	-.181	.247
		Equal variances not assumed			.346	206	.730	.033	.095	-.154	.220
Group 5 and 6	IRI (Pre-flood)	Equal variances assumed	7.170	.008	-2.310	120	.023	-.579	.250	-1.074	-.083
		Equal variances not assumed			-1.991	45	.053	-.579	.291	-1.164	.007
	IRI (Post-flood)	Equal variances assumed	7.271	.008	-2.713	120	.008	-1.106	.408	-1.913	-.299
		Equal variances not assumed			-1.863	35	.071	-1.106	.594	-2.311	.099
	$\Delta IRI_{post\text{-}flood}$	Equal variances assumed	9.986	.002	-1.716	120	.089	-.527	.307	-1.136	.081
		Equal variances not assumed			-1.116	34	.272	-.527	.472	-1.488	.433

These pavement sections consist of roads across Queensland which were rehabilitated at different times from 2010 to 2015. There were significant increases in rutting and roughness values after 2011. Overall, rapid increases in rutting values, and in some cases, rapid increases in roughness were observed. An extensive statistical analysis of flood affected Queensland roads shows significant increases in rutting and roughness following the flooding events.

This study demonstrated that there were significant correlations between pre-flood rutting values and increased road deterioration after floods. Roads with lower pre-flood rutting were highly likely to have lower post-flood rutting and roughness. Roads with higher pre-flood rutting were highly likely to have higher post-flood rutting and roughness and are more likely to deteriorate and increase the cost of road rehabilitation. Pavements were categorized based on their pre-flood rutting values. A road with low pre-flood rutting is highly likely to have a lower increase in post flood rutting and roughness than a road with high pre-flood rutting (road in poor condition). This study indicates that pre-flood rutting should be considered for modelling rapid deterioration of rutting of flood affected pavements. Hence, pre-flood rutting was included in modelling the deterioration of rutting of flood affected pavements presented later in this chapter.

Flooding had a great impact on accelerating the deterioration of surface conditions, such as rutting and roughness of roads. It is evident that flooding events in the last six years increased the cost of rehabilitation due to the increased deterioration of pavements. Individual roads were also analysed in this chapter (refer to Sections 6.5 to 6.11) to assess the impact of flooding on pavements.

6.5 HISTORY OF ROADS INCLUDED IN ANALYSIS AND MODELLING

Pavement history of roads that were analysed individually and included in the modelling is given in Table 6.14. Table 6.15 provides the mean and standard deviation of roughness and rutting data relating to these individual roads.

Table 6.14: Pavement History of TMR, Queensland Roads included in detailed analysis and modelling

Road Id	Road Name	Name of Council	District Name	Region Name	No of sections analysed	No of sections in Modelling	Rehab Completed
194	Rockhampton - Emu Park Road	Livingstone Shire Council	Fitzroy	Central Queensland	47	15	03-Nov-11
197	Western Yeppoon Emu Park Road	Livingstone Shire Council	Fitzroy	Central Queensland	17	12	12-Oct-11
188	Bajool - Pt Alma Road	Rockhampton Regional Council	Fitzroy	Central Queensland	131	25	28-May-12
46A	Gladstone – Biloela in Dawson Highway	Gladstone Regional Council	Fitzroy	Central Queensland	112	68	4-Oct-12 & 13-Dec-13
10E	Benaraby – Rockhampton in Bruce Highway	Gladstone Regional Council	Fitzroy	Central Queensland	-	9	31-Mar-14 & 15-Apr-14
10E	Benaraby – Rockhampton in Bruce Highway	Rockhampton Regional Council	Fitzroy	Central Queensland	-	20	9-Sep-13
10F	Rockhampton-St Lawrence in Bruce Highway	Rockhampton Regional Council	Fitzroy	Central Queensland	-	1	13-Dec-13
10G	St Lawrence – Mackay in Bruce Highway	Isaac Regional Council	Mackay/ Whitsunday	Central Queensland	-	5	19-Dec-12 & 2-Oct-13
10G	St Lawrence – Mackay in Bruce Highway	Mackay Regional Council	Mackay/ Whitsunday	Central Queensland	-	2	9-Nov-13 & 3-Jul-13
10H	Mackay – Proserpine in Bruce Highway	Whitsunday Regional Council	Mackay/ Whitsunday	Central Queensland	-	9	19-Dec-12 & 1-Oct-13
10K	Bowen – Ayr in Bruce Highway	Whitsunday Regional Council	Mackay/ Whitsunday	Central Queensland	-	2	17-Sep-12 & 9-Oct-13
10L	Ayr – Townsville in Bruce Highway	Burdekin Shire Council	Northern	North Queensland	-	5	25-Oct-13
10L	Ayr – Townsville in Bruce Highway	Townsville City Council	Northern	North Queensland	-	5	3-Jul-12
303	Rosewood Marburg Road	Ipswich City Council	Metropolitan	Metropolita n	22	6	21-Nov-12
305	Rosewood - Warill View Road	Ipswich City Council	Metropolitan	Metropolita n	10	9	27-Nov-12
308	Rosewood - Laidley Road	Ipswich City Council	Metropolitan	Metropolita n	10	7	12-Mar-13
18D	Miles Roma Road in Warrego Highway	Western Downs Regional Council	Darling Downs	Downs South West	495	18	4-Dec-12
Total						218	

Table 6.15: Mean and standard deviation of rutting and roughness of TMR, Queensland Roads included in analysis

Road Id	Road Name	Statistical Parameter	Rutting (mm)							Roughness (IRI)						
			2009	2010	2011	2012	2013	2014	2015	2009	2010	2011	2012	2013	2014	2015
194	Rockhampton - Emu Park Road	Mean	7.3	12.6	4.5	4.8	5.9	6.7		2.77	2.73	1.81	1.89	1.90	1.90	
		Std	2.60	4.20	1.77	2.02	3.22	3.40		0.79	0.83	0.39	0.49	0.48	0.49	
197	Western Yeppoon Emu Park Road	Mean	5.6	12.4	7.2	7.6	8.9	10.2		1.59	1.72	1.91	1.91	2.07	2.08	
		Std	1.67	2.51	2.23	2.14	3.84	4.67		0.14	0.19	0.19	0.38	0.43	0.51	
303	Rosewood Marburg Road	Mean		6.5	7.8	7.4	4.3	4.1			4.15	4.33	4.64	2.03	2.16	
		Std		2.38	3.19	3.26	1.02	1.13			1.02	0.98	1.27	0.48	0.41	
305	Rosewood - Warill View Road	Mean		7.8	15.0	14.2	5.4	5.2	5.0		2.81	3.38	3.33	2.11	2.01	1.95
		Std		2.63	2.59	2.85	2.12	2.62	1.83		0.60	0.67	0.70	0.59	0.62	0.57
308	Rosewood - Laidley Road	Mean		8.8	11.7	12.2	3.7	1.6			2.78	3.17	3.33	2.29	1.65	
		Std		2.50	3.24	3.84	0.63	0.38			0.77	0.69	0.90	0.35	0.13	
188	Bajool - Pt Alma Road	Mean	4.9	8.6	4.7	4.6	4.9	5.3		2.38	2.43	2.17	2.14	2.13	2.08	
		Std	3.32	4.64	1.60	1.43	1.96	1.89		0.71	0.77	0.59	0.57	0.65	0.59	
46A	Gladstone Biloela in Dawson Highway	Mean	6.0	13.4	6.9	4.4	6.5	7.2		2.06	2.17	2.37	2.13	2.29	1.95	
		Std	3.22	5.98	4.17	0.84	1.74	3.21		0.59	0.62	0.57	0.58	0.78	0.46	
18D	Miles Roma Road of Warrego Highway	Mean		5.1	6.1	5.4	4.3	5.0			2.79	2.75	2.60	1.97	2.20	
		Std		2.31	2.87	3.10	2.19	2.38			0.85	0.82	1.04	0.64	0.69	

6.6 ROCKHAMPTON EMU PARK ROAD, LIVINGSTONE SHIRE COUNCIL

Detailed data for each layer of the different pavement sections included in the analysis for the Rockhampton-Emu Park Road (Road ID: 194) in Livingstone Shire Council, Central Queensland were recorded from the pavement history file and are summarised in Tables 6.16 and 6.17.

Table 6.16: Pavement history of the sections (37.2, 39.9, 41.2 and 42.3) of Rockhampton-Emu Park Road included in modelling

Start ID	Layer	Description	Layer Depth (mm)	Rehab Date
All	L1	PMB Spray Seal	10	03-Nov-2011
All	L2	Bitumen Spray Seal	16	03-Nov-2011
37.2	L3	Bitumen Geotextile Seal	10	1-Apr-2005
39.9 41.2 42.3	L3	Bitumen Spray Seal	10	23-May-2003 23-May-2003 1-Apr-2005
All	L4	Bitumen Spray Seal	7	26-Jun-1997
All	L5	Spray Seal - Quality Unknown	20	1-Jan-1985/1984
All	L6	Granular, CTB or AC - Quality Unknown	100	1-Jan-1965/1960
All	L7	Subgrade - Quality Unknown	100	1-Jan-1965/1960
Note: PMB –Polymer modified binder				

Table 6.17: Pavement history of the sections (37.5, 39 to 39.7, 40.4 to 40.7 and 42.5) of Rockhampton-Emu Park Road included in modelling:

Layer	Description	Layer Depth (mm)	Rehab Date
L1	PMB Spray Seal	10	03-Nov-2011
L2	Bitumen Spray Seal	16	03-Nov-2011
L3	Cement Stabilised Granular-Modified	200	03-Nov-2011
L4	Subgrade - Quality Unknown	100	1-Jan-1965
L4 for 42.5	Granular, CTB or AC - Quality Unknown	100	1-Jan-1960
L5 for 42.5	Subgrade - Quality Unknown	100	1-Jan-1960
Note: PMB –Polymer modified binder			

The rutting and roughness plot for rehabilitated sections of Rockhampton Emu Park Road are shown in Figures 6.1 and 6.2. Post-flood data for the Rockhampton Emu Park Road were collected on 19 July, 2010, almost six months after the heavy rainfall and flooding event that occurred in the area in February, 2010. There were significant increases in post-flood rutting following the flooding event. However, as seen in Figure 6.2 there were not very high increases in roughness values.

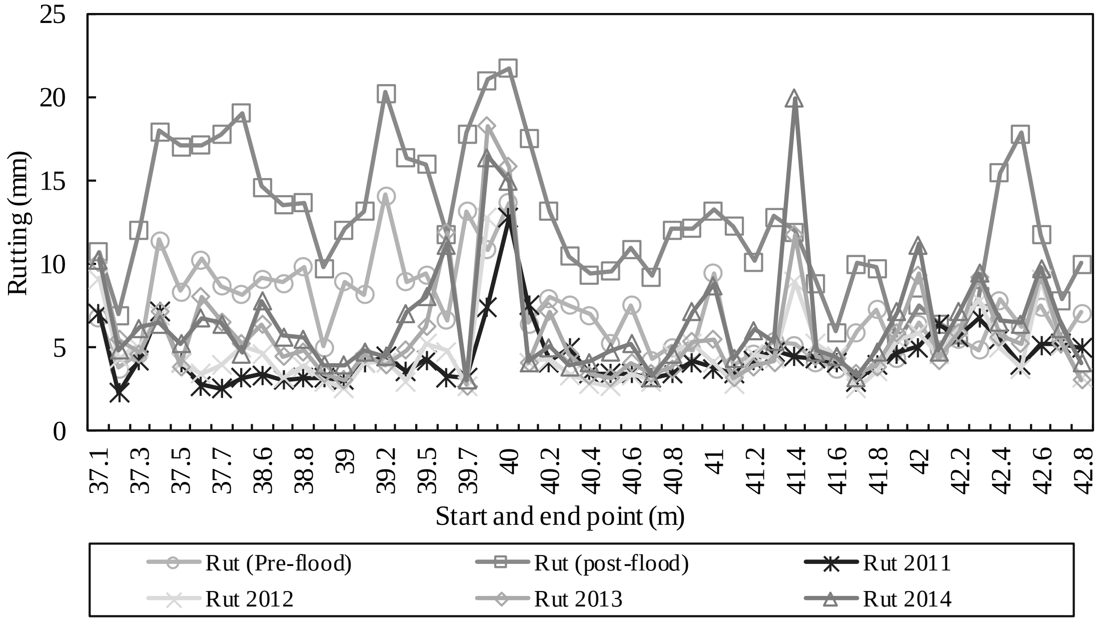


Figure 6.1: Rutting plot for flood affected rehabilitated sections of Rockhampton Emu Park Road (ID: 194), Livingstone Shire Council, Central Queensland

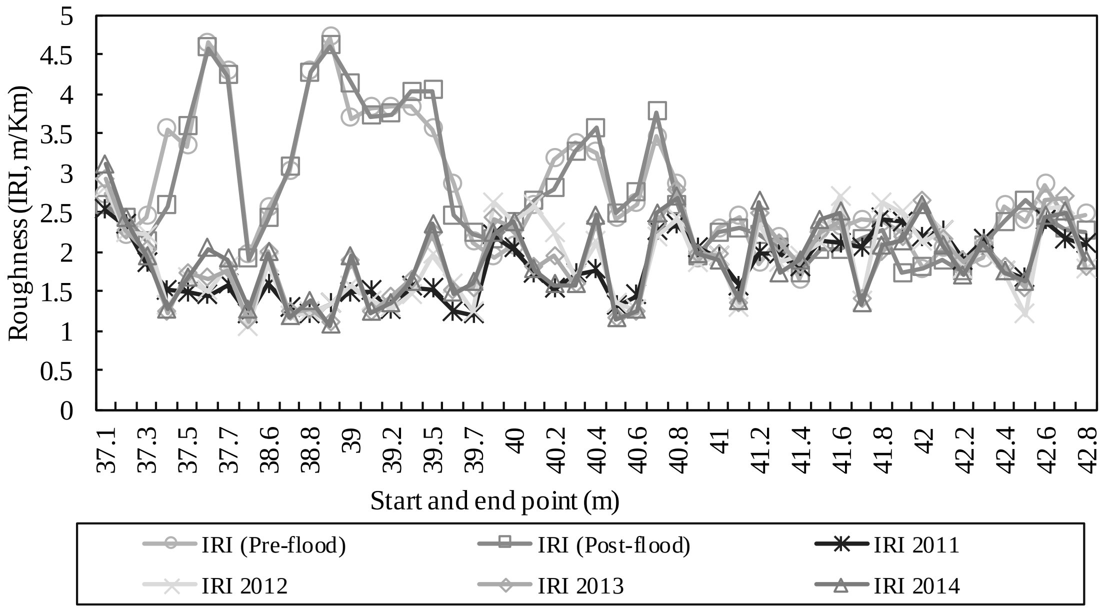


Figure 6.2: Roughness plot for flood affected rehabilitated sections of Rockhampton Emu Park Road (ID: 194), Livingstone Shire Council, Central Queensland

The rehabilitation work on the Rockhampton Emu Park Road was completed in November 2011. As a result of the rehabilitation, both rutting and roughness data, collected from 2011 to 2014, showed significant improvement. However, a few sections again deteriorated in 2013 and 2014. The heavy rainfall event (nearly 300 mm of rain) in parts of the Capricorn Coast, from the morning of 25 March to 26 March, 2014, flooded around 50 roads in the Rockhampton, Livingstone and Gladstone Council areas (Rollo and Robinson 2014). The monthly rainfall data (Table 6.2) also confirms that there were heavy rainfall events in the area in January and March of 2013 and February and March of 2014 may be one of the causes of increasing rutting values in some sections of the road in 2013 and 2014. Three sections were identified as critical sections for the analysis: the data for roughness and rutting have been plotted in Figures 6.3, 6.4 and 6.5. The rates of increase and decrease in the rutting and roughness values of the three critical sections are presented in Table 6.18.

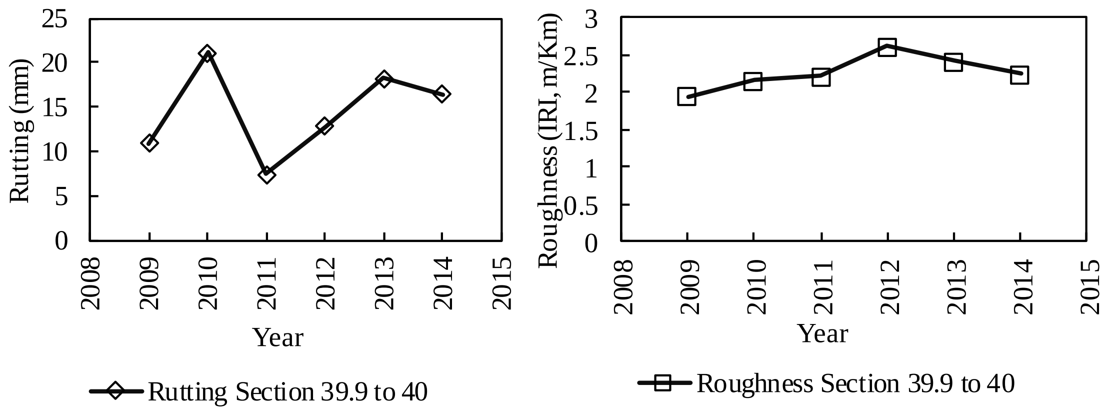


Figure 6.3: Loss of rutting and roughness in section 39.9-40 of Rockhampton Emu Park Road, Livingstone Shire Council, Central Queensland

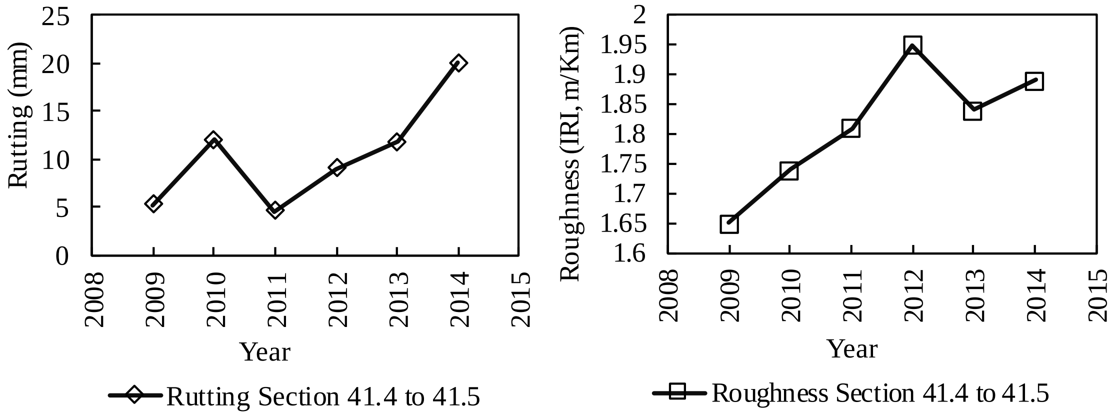


Figure 6.4: Loss of rutting and roughness in section 41.4-41.5 of Rockhampton Emu Park Road, Livingstone Shire Council, Central Queensland

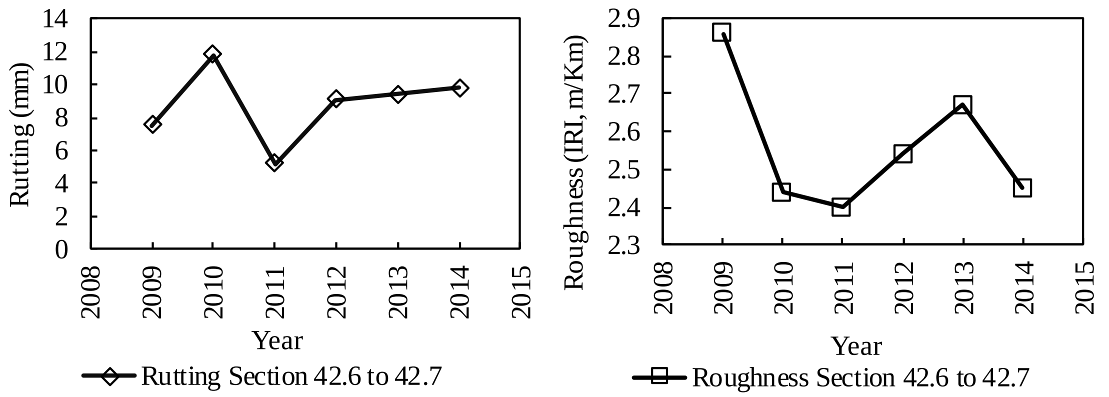


Figure 6.5: Loss of rutting and roughness in section 42.6-42.7 of Rockhampton Emu Park Road, Livingstone Shire Council, Central Queensland

Table 6.18: Rate of increase/decrease in rutting and roughness values every year for three critical sections of Rockhampton Emu Park Road

Road Section ID	**Increase/decrease in Rutting (mm/year)**					**Increase/decrease in Roughness (IRI/year)**				
	2010	**2011**	**2012**	**2013**	**2014**	**2010**	**2011**	**2012**	**2013**	**2014**
Section 1 (39.9 to 40)	10.2	-13.6	5.3	5.5	1.8	0.22	0.5	0.41	-0.2	0.18
Section 2 (41.4 to 41.5)	6.8	-7.5	4.5	2.8	8.2	0.09	0.07	0.14	-0.11	0.05
Section 3 (42.6 to 42.7)	4.3	-6.6	3.9	0.3	0.5	-0.42	-0.04	0.14	0.13	-0.22
Note: Positive value means an increase and negative value means decrease in the rate of rutting and roughness values										

6.7 WESTERN YEPPOON EMU PARK ROAD, LIVINGSTONE SHIRE COUNCIL

Detailed data for each layer of the different pavement sections included in the analysis for the Western Yeppoon Emu Park Road (Road ID: 197) in Livingstone Shire Council, Central Queensland were recorded from the pavement history file and are shown in Table 6.19.

Table 6.19: Pavement history of the sections of Western Yeppoon-Emu Park Road included in modelling

Layer	Description	Layer Depth (mm)	Rehab Date
L1	PMB Spray Seal	10	22-Oct-2011
L2	Bitumen Spray Seal	14	22-Oct-2011
L3	Cement Stabilised Granular-Modified	200	22-Oct-2011
L4	Category 1 Cement Stabilised Granular	80	30-Jun-2005
L5	Subgrade - Quality Unknown	100	30-Jun-2005
L6	Natural Soil Subgrade	100	30-Jun-2005
Design ESA: 5×10^{6}. Note: PMB –Polymer modified binder			

Post-flood data for the Western Yeppoon-Emu Park Road were collected on 19 July, 2010, almost six months after the heavy rainfall and flooding event that occurred in the area in February, 2010. The rutting and roughness data from 2009-2014 for the selected flood affected sections (17 sections) of the Western Yeppoon-Emu Park Road have been plotted in Figures 6.6 and 6.7. The rehabilitation work of these sections was completed in October 2011. These graphs clearly indicate that there were increases in the rutting values after the flooding event in 2010. Although, the rutting values in 2012, 2013 and 2014, show some improvement, there was an increase in the roughness values in some sections in 2014 and 2015. However, post-flood roughness values of these sections were not as highly impacted as rutting values.

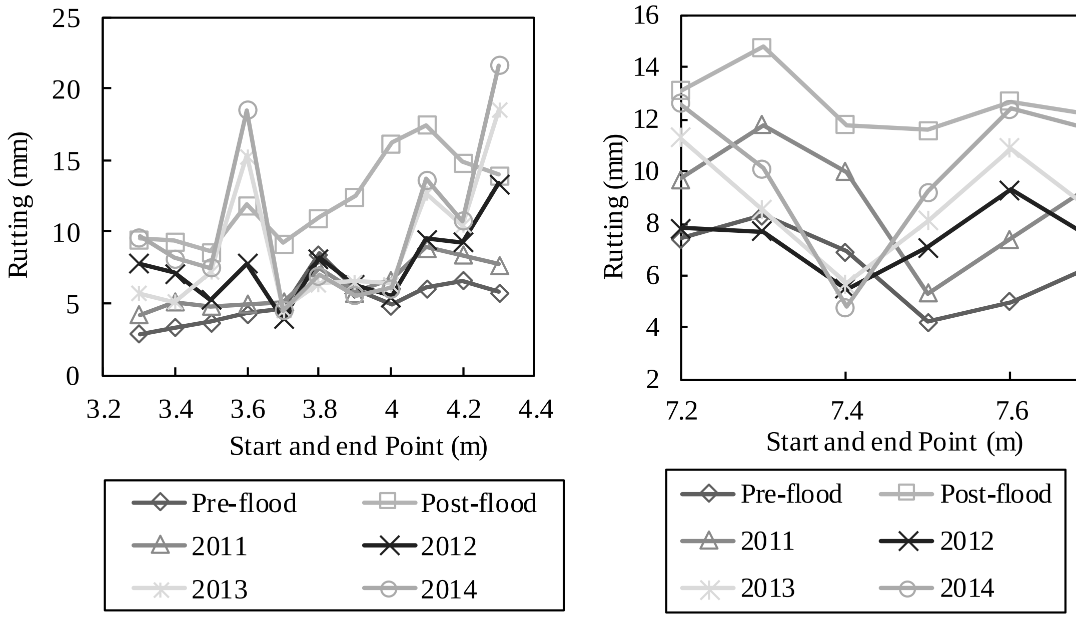


Figure 6.6: Rutting plot for flood affected rehabilitated sections of Western Yeppoon-Emu Park Road, Livingstone Shire Council, Central Queensland

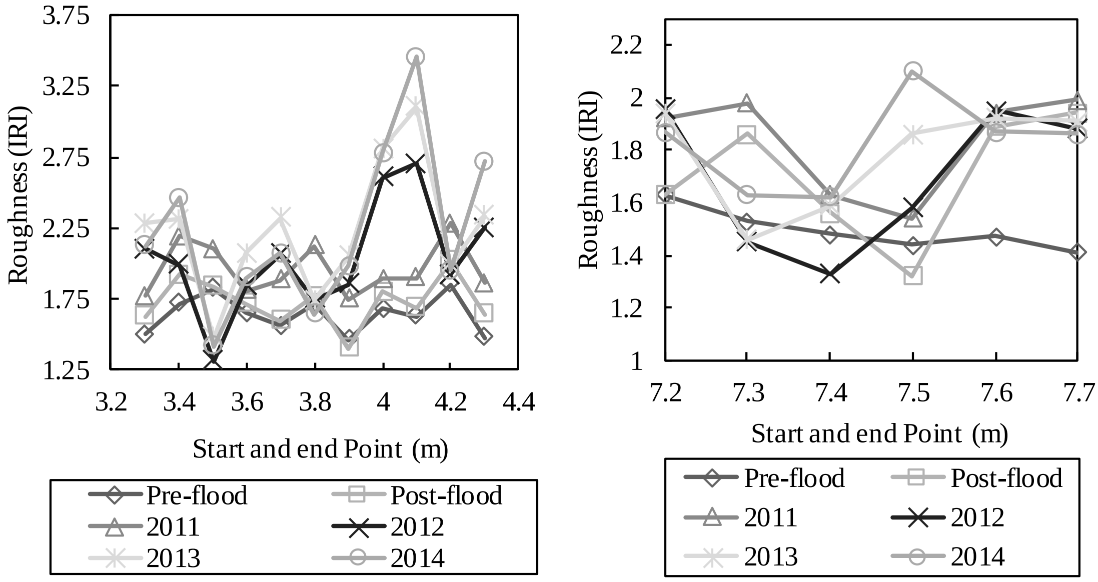


Figure 6.7: Roughness plot for flood affected rehabilitated sections of Western Yeppoon-Emu Park Road, Livingstone Shire Council, Central Queensland

6.8 ROSEWOOD MARBURG ROAD, IPSWICH CITY COUNCIL

Detailed data for each layer of the different pavement sections included in the analysis for the Rosewood Marburg Road (Road ID: 303) in the Ipswich City Council area were recorded from the pavement history file and are shown in Table 6.20. This road was flooded during the January 2011 flooding event.

Table 6.20: Pavement History of Rosewood Marburg Road (ID 303)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	PMB Spray Seal	10	21-Nov-2012
L2	PMB Spray Seal	14	21-Nov-2012
L3	Primerseal	7/14	21-Nov-2012
L4	Cement Stabilised Granular-Modified	300	21-Nov-2012
L5	Granular - Quality Unknown	144	1-Jan-1964
L6	Subgrade - Quality Unknown	100	1-Jan-1964

The rutting and roughness data for flood affected sections of Rosewood Marburg Road, Ipswich have been plotted in Figures 6.8 and 6.9. The rehabilitation of this road was completed in November 2012. There were significant increases in post-flood rutting values in many pavement sections. There were also increases in post-flood roughness values. Some sections also continued to have increased rutting and roughness until 2012. Rutting and roughness values significantly decreased after the completion of the rehabilitation works in 2012. Rutting and roughness plots of 2013 and 2014 show significant decreases in rutting and roughness values.

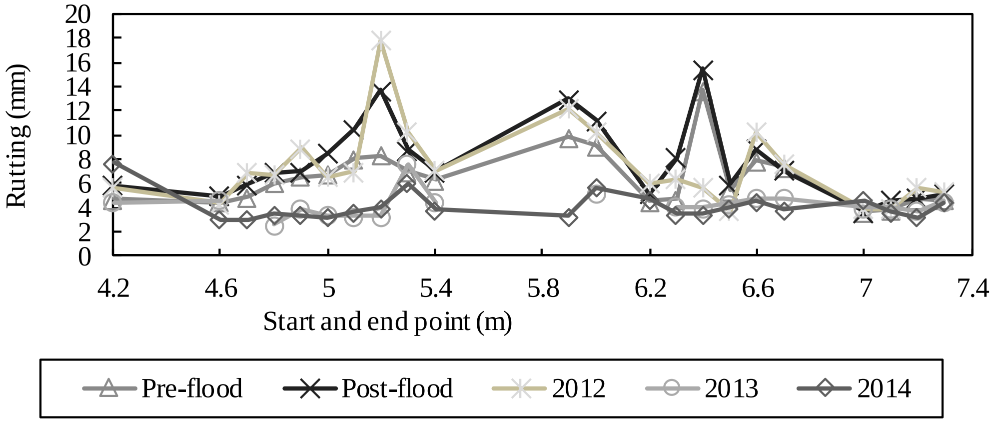


Figure 6.8: Rutting plot for flood affected rehabilitated sections of Rosewood Marburg Road, Ipswich

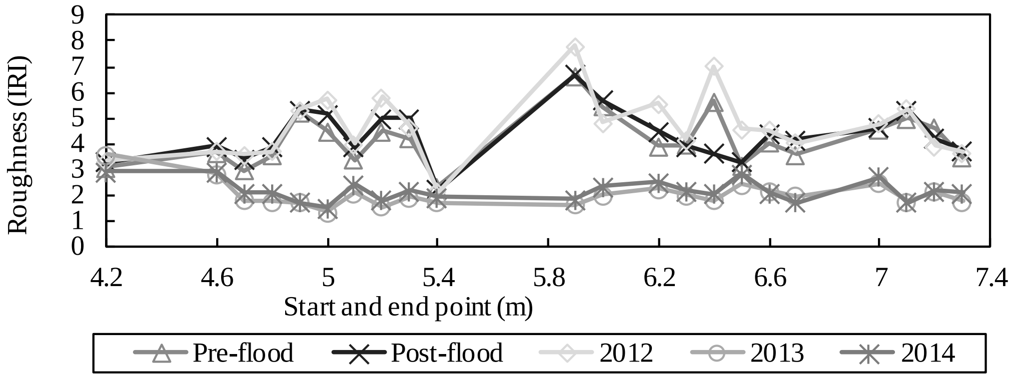


Figure 6.9: Roughness plot for flood affected rehabilitated sections of Rosewood Marburg Road, Ipswich

6.8.1 *Rosewood Warill View Road, Ipswich City Council*

Detailed data for each layer of the different pavement sections included in the analysis for the Rosewood Warill View Road (Road ID: 305) in Ipswich were recorded from the pavement history file and are shown in Table 6.21. This road was flooded during the January 2011 flooding event.

Table 6.21: Pavement history of Rosewood Warill View Road (ID 305)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	PMB Spray Seal	10	27-Nov-2012
L2	PMB Spray Seal	14	27-Nov-2012
L3	Primerseal	7	27-Nov-2012
L4	Cement Stabilised Granular-Modified	250	27-Nov-2012
L5	Granular - Quality Unknown	100	11-Dec-1986
L6	Natural Soil Subgrade	100	11-Dec-1986

The rutting and roughness data for rehabilitated flood affected sections (10 sections) of the Rosewood Warill View Road have been plotted in Figures 6.10 and 6.11. There were significant increases in post-flood rutting values in many pavement sections like Rosewood Marburg Road. There were increases in post-flood roughness values. Some sections also continued to increase in rutting and roughness values until 2012. Rutting and roughness values significantly decreased after the completion of the rehabilitation works in November 2012.

Rutting and roughness data from 2013 and 2014 show significant decreases in rutting and roughness values.

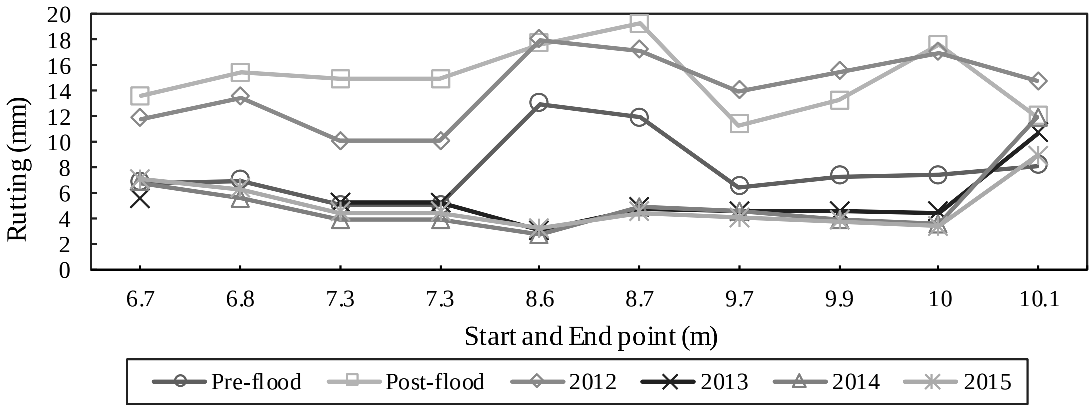


Figure 6.10: Rutting plot for flood affected rehabilitated sections of Rosewood Warill View Road, Ipswich

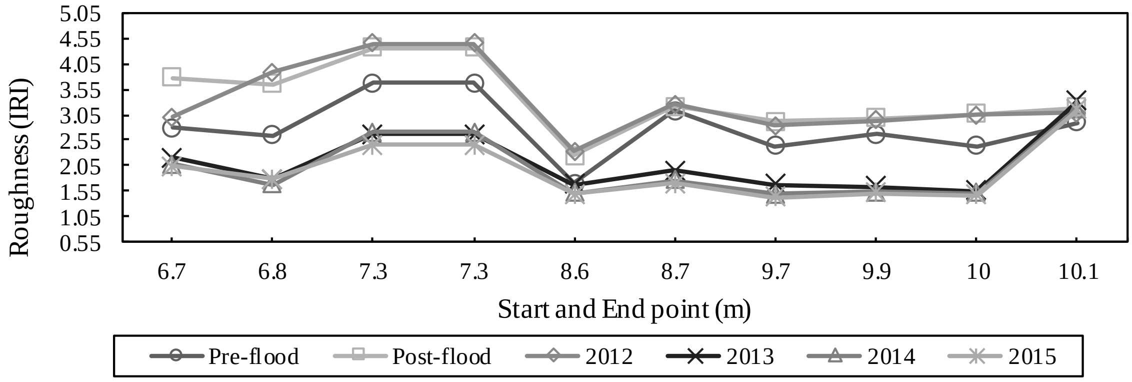


Figure 6.11: Roughness plot for flood affected rehabilitated sections of Rosewood Warill View Road, Ipswich

6.8.2 Rosewood Laidley Road, Ipswich City Council

Detailed data for each layer of the different pavement sections included in the analysis for the Rosewood Laidley Road (Road ID: 308) in Ipswich were recorded from the pavement history file and are shown in Table 6.22. This road was flooded during the January 2011 flooding event.

Table 6.22: Pavement History of Rosewood Laidley Road (ID 308)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	Bitumen Dense Graded Asphalt	55	3-Dec-2012
L2	PMB Spray Seal	10	3-Dec-2012
L3	Primerseal	7	3-Dec-2012
L4	Cement Stabilised Granular-Modified	270	3-Dec-2012
L5	Granular - Quality Unknown	80	1-Jan-1967

The data for rutting and roughness in rehabilitated flood affected sections (11 sections) of Rosewood Laidley Road have been plotted in Figures 6.10 and 6.11. The rehabilitation of this road was completed in December 2012. There were increases in rutting values in and roughness values in 2011 (post-flood) and also for some sections in 2012. However, rutting and roughness values decreased significantly after the completion of the rehabilitation works in 2012.

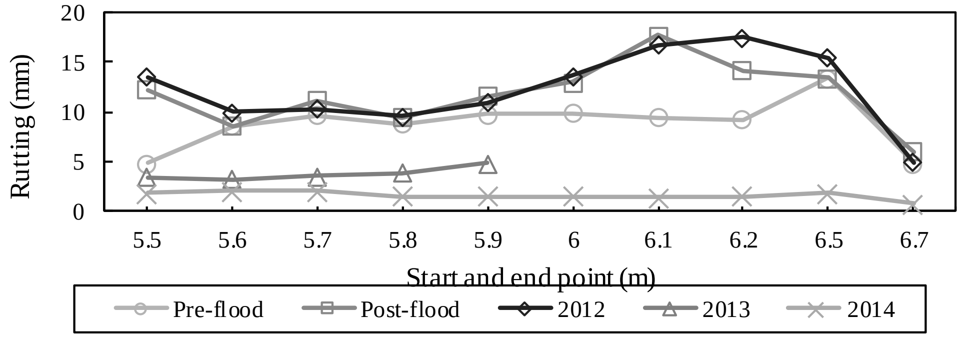


Figure 6.12: Rutting plot for flood affected rehabilitated sections of Rosewood Warill View Road, Ipswich

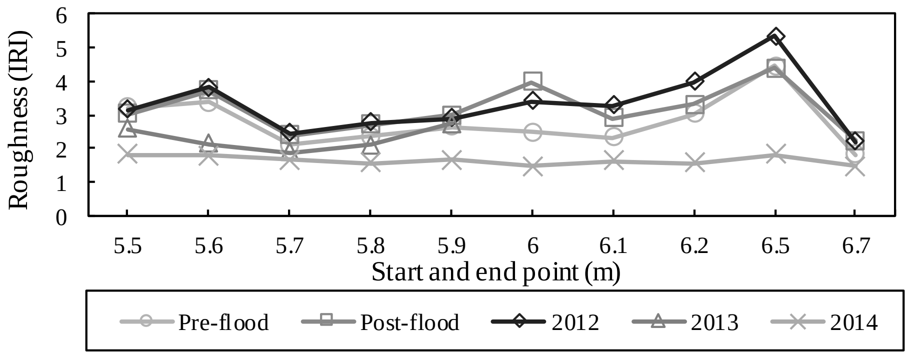


Figure 6.13: Roughness plot for flood affected sections of Rosewood Warill View Road, Ipswich

6.9 BAJOOL PORT ALMA ROAD IN ROCKHAMPTON REGIONAL COUNCIL

Detailed data for each layer of the different pavement sections included in the analysis for the Bajool Port Alma Road (ID 188) in Rockhampton, Central Queensland were recorded from the pavement history file and are shown in Table 6.23. For this road, layer 1 and layer 2 were Bitumen Spray Seal. However, as date and layer depth varies for different sections a detailed list is needed.

Table 6.23: Pavement history of Bajool Port Alma Road (Road ID 188)

Layer	**Description**	**Layer Depth (mm)**	**Rehab Date**
L1	Bitumen Spray Seal	10	10-Mar-2005/ 28-May-2012/ 30-Jul-2006/ 1-Jan-2008
L2	Bitumen Spray Seal	14 / 10	15-Feb-2002/ 28-May-2012/ 30-May-2004/ 30-Jul-2006
L3	Bitumen Spray Seal	16/10	1-Jan-1977/ 28-May-2012/ 30-May-2004/ 1-Feb-1993
L3	Cement Stabilised Granular-Modified	200	
L4	Granular, CTB or AC - Quality Unknown	200	1-Jan-1977/ 1-Jan-1967/ 15-May-2000/ 1-Jan-1974/ 1-Feb-1993
L4	Subgrade - Quality Unknown	100	
L4	Spray Seal - Quality Unknown	20	
L4	Standard Granular	150	
L5	Subgrade - Quality Unknown	100	1-Jan-1967/ 15-May-200
L5	Granular, CTB or AC - Quality Unknown	100	1-Jan-1974
L5	Spray Seal - Quality Unknown	20	1-Jan-1986
L6	Natural Soil Subgrade	100	1-Jan-1977/ 15-May-2000
L6	Subgrade - Quality Unknown	100	1-Jan-1974
L6	Granular, CTB or AC - Quality Unknown	100	1-Jan-1974
L7 (one section)	Subgrade - Quality Unknown	100	1-Jan-1974
Design ESA: 1×10^6			

The data for the rutting and roughness for rehabilitated flood affected sections (131 sections) of the Bajool Port Alma Road have been plotted in Figures 6.14 and 6.15. The rehabilitation of this road was completed in May 2012. There were increases in rutting values in and roughness values in 2010 (post-flood) and also for some sections in 2011. However, rutting and roughness values decreased significantly after the completion of the rehabilitation works in 2012.

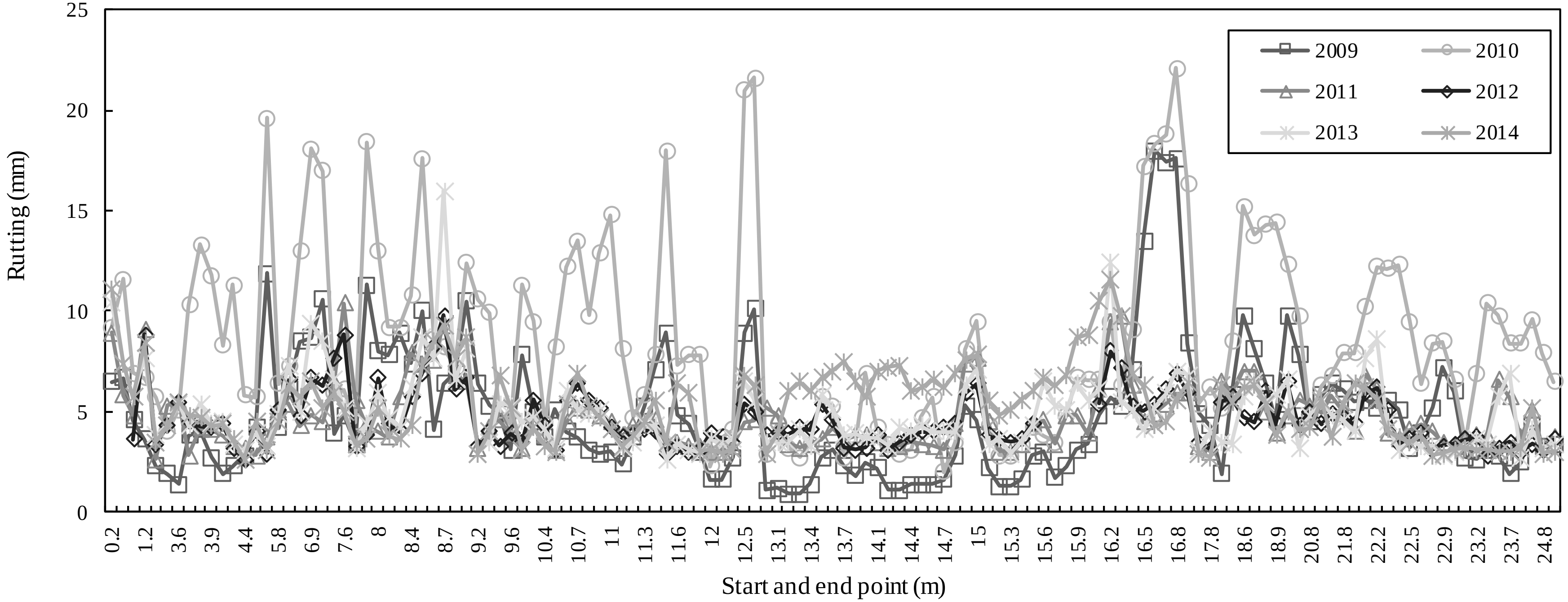


Figure 6.14: Rutting plot for flood affected rehabilitated sections of Bajool Port Alma Road, Rockhampton Regional Council

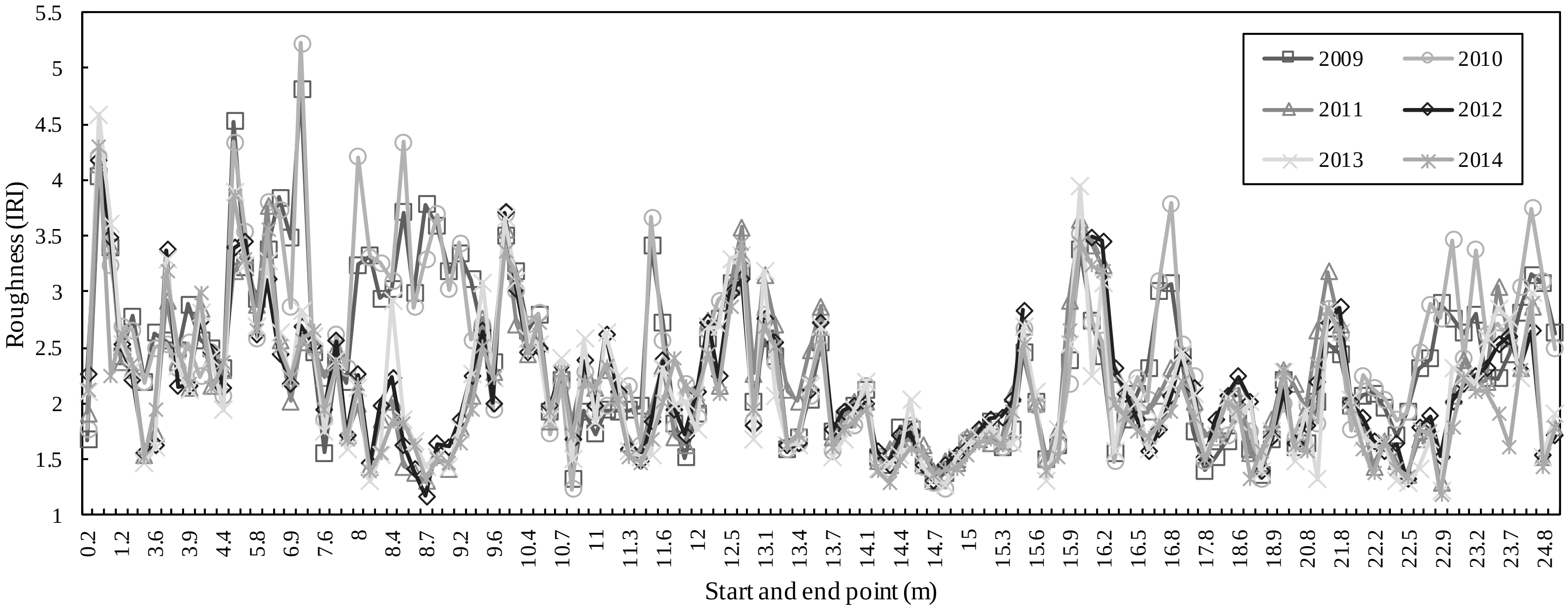


Figure 6.15: Roughness plot for flood affected rehabilitated sections of Bajool Port Alma Road, Rockhampton Regional Council

6.10 GLADSTONE BILOELA IN DAWSON HIGHWAY

Detailed data for each layer of the different pavement sections included in the analysis for the Gladstone Biloela in Dawson Highway (Road ID: 46A) in Gladstone, Central Queensland were recorded from the pavement history file and are shown in Table 6.27. The data for the rutting and roughness for rehabilitated flood affected sections (112 sections) of the Gladstone Biloela in Dawson Highway have been plotted in Figures 6.16 and 6.17. The rehabilitation of some sections of this road was completed in October 2012 and other sections in December 2013. There were increases in rutting values in and roughness values in 2010 (post-flood).

Table 6.24: Pavement history of Gladstone Biloela in Dawson Highway (Road ID 46A)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	Bitumen Dense Graded Asphalt	50	4-Oct-2012
L1	Bitumen Spray Seal	16	13-Dec-2013
L1	PMB Spray Seal	10	13-Dec-2013
L2	PMB Spray Seal	10	4-Oct-2012
L2	Bitumen Spray Seal	7/16	1-Jun-2005/ 5-Nov-2005
L2	Cement Stabilised Granular-Modified	200	13-Dec-2013
L2	Standard Granular	225	13-Dec-2013
L3	Bitumen Dense Graded Asphalt	275	4-Oct-2012
L3	Bitumen Spray Seal	16	1-Jun-2005
L3	Standard Granular	125	30-Nov-2007
L3	Category 1 Cement Stabilised Granular	150	29-Jul-2003
L3	Primerseal	7	5-Nov-2005
L3	Cement Stabilised Granular-Modified	200	13-Dec-2013
L4	Cement Stabilised Granular-Modified	175	4-Oct-2012
L4	Primerseal	7	1-Jun-2005
L4	Standard Granular	125 150	30-Nov-2007/ 29-Jul-2003
L4	Nonstandard Granular	200	30-Nov-2007
L4	Category 1 Cement Stabilised Granular	150	5-Nov-2005
L4	Granular, CTB or AC - Quality Unknown	140	1-Jan-1952
L4	Cement Stabilised Granular-Modified	75	30-Nov-2007
L4	Natural Soil Subgrade	100	13-Dec-2013
L5	Subgrade - Quality Unknown	210	1-Jan-1946/ 1-Jan-1953
L5	Category 1 Cement Stabilised Granular	150	1-Jun-2005
L5	Natural Soil Subgrade	100	30-Nov-2007
L5	Granular, CTB or AC - Quality Unknown	100	1-Jan-1952
L6	Natural Soil Subgrade	100	30-Nov-2007
L6	Subgrade - Quality Unknown	75	1-Jan-1946/ 1-Jan-1953
Design ESA: 1×10^6			

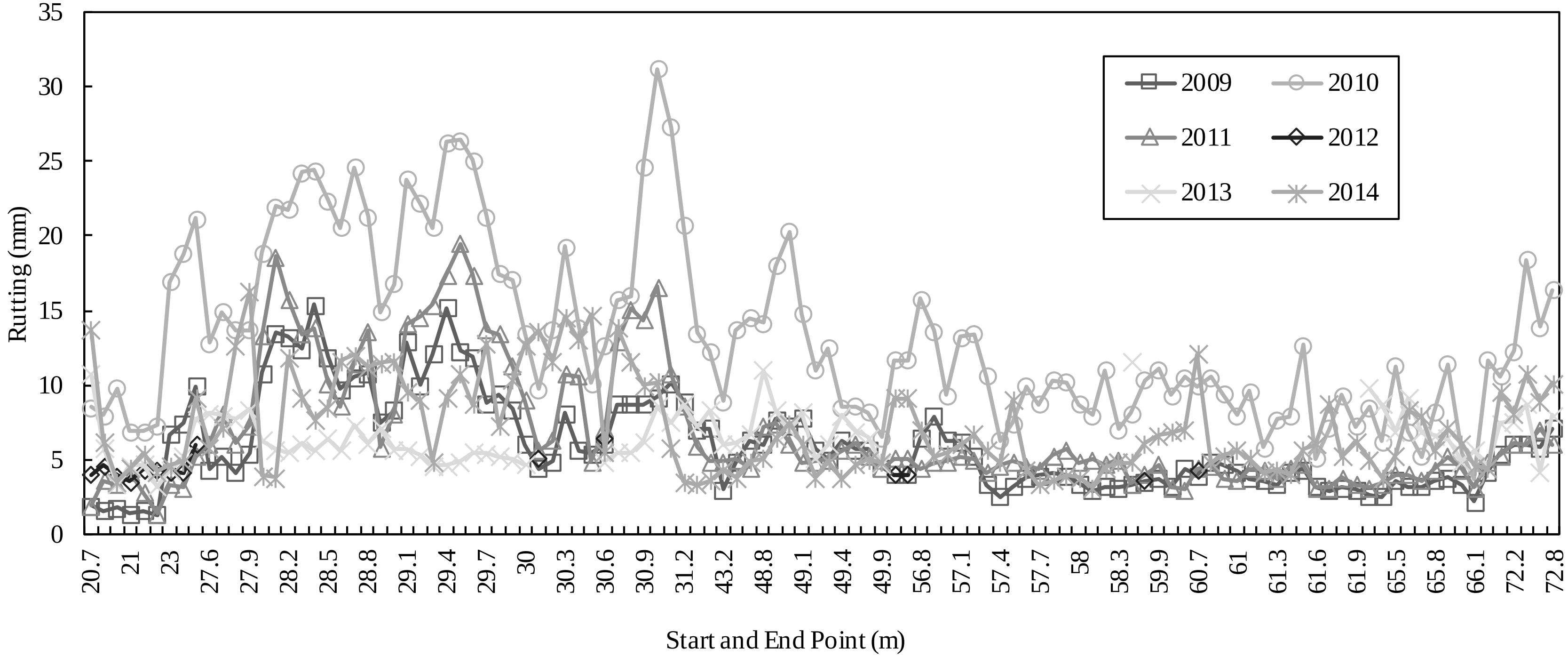


Figure 6.16: Rutting plot for flood affected rehabilitated sections of Gladstone Biloela in Dawson Highway, Gladstone

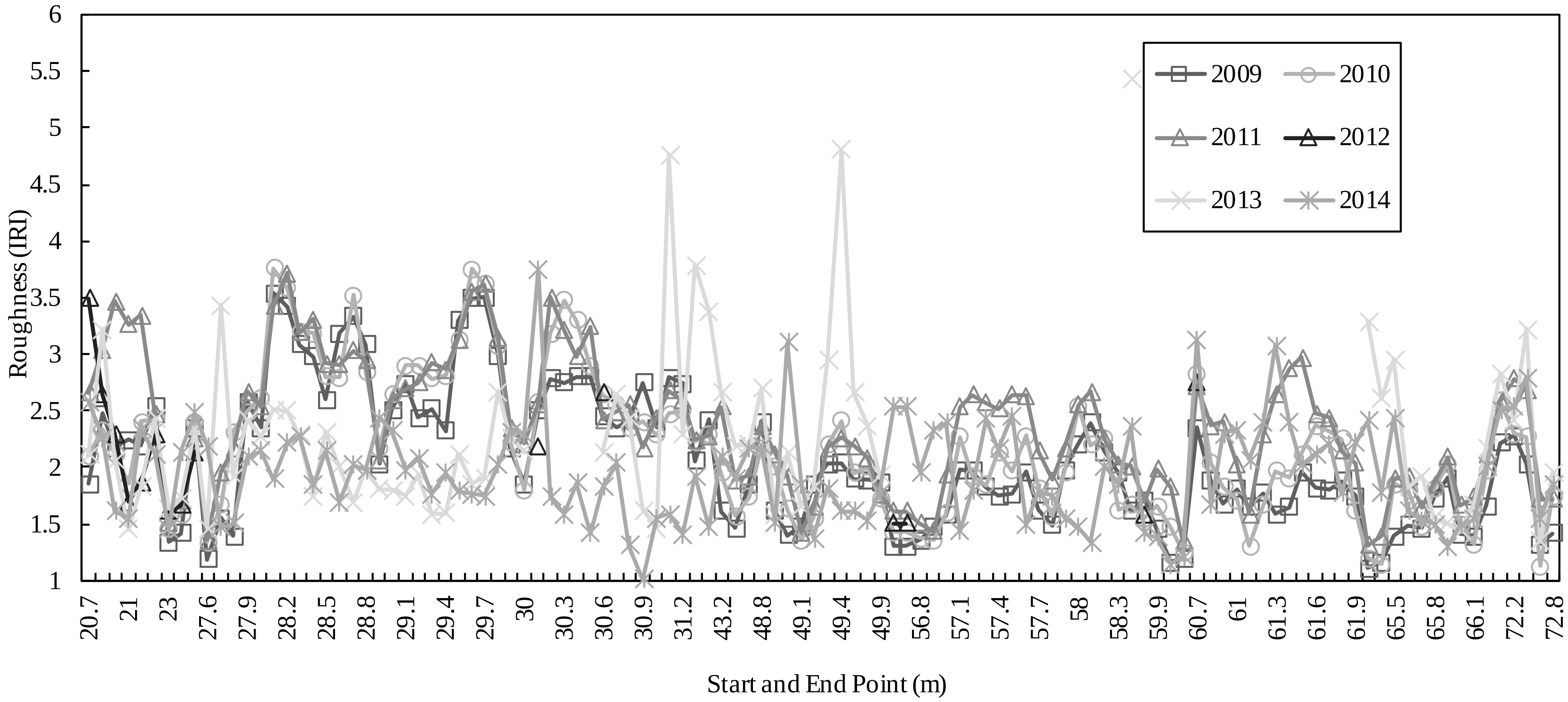


Figure 6.17: Roughness plot for flood affected rehabilitated sections of Gladstone Biloela in Dawson Highway, Gladstone

6.11 MILES ROMA WARREGO HIGHWAY

A large section of the Miles Roma Road of Warrego Highway (Road ID 18D) in Western Downs, Darling Downs was damaged due to the flood and later rehabilitated. Among the 975 rehabilitated sections, 495 sections had post-flood rutting greater than or equal to pre-flood rutting. Due to the large sample size, graphical representation of this road was not done. However, data from the analysed sections are included in Appendix B. Detailed data for each layer of the different pavement sections included in the analysis for the Miles Roma Road of Warrego Highway were recorded from the pavement history file and are shown in Tables 6.25 to 6.27.

Table 6.25: Pavement history of Miles Roma Road of Warrego Highway (Road ID 18D, pavement sections start Id 22.3 to 24.6)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	Spray Seal-Emulsion	7	21-Dec-2012
L2	PMB Spray Seal	14	21-Dec-2012
L3	Bitumen Dense Graded Asphalt	35	21-Dec-2012
L4	PMB Spray Seal	10	2-Nov-2005
L5	Bitumen Spray Seal	10	15-Dec-1996
L6	Bitumen Spray Seal	10	6-Oct-1989
L7	Bitumen Spray Seal	10	3-Sep-1987
L8	Standard Granular	150	3-Sep-1987
L9	Subgrade - Quality Unknown	100	10-Aug-66
L10	Subgrade - Quality Unknown	100	10-Aug-66

Table 6.26: Pavement history of Miles Roma Road of Warrego Highway (Road ID 18D, pavement sections start Id 25.4, 25.6, 30.8, 30.9 and 34.9)

Layer	Description	Layer Depth (cm)	Rehab Date
L1	Spray Seal-Emulsion	7	21-Dec-2012
L2	Bitumen Geotextile Seal	14	21-Dec-2012
L3	Primerseal	10	21-Dec-2012
L4	Cement Stabilised Granular-Modified	200	21-Dec-2012
L5	Subgrade - Quality Unknown	100	21-Dec-2012

Table 6.27: Pavement history of Miles Roma Road of Warrego Highway (Road ID 18D, pavement sections start Id 91.1, 91.2 and 91.3)

Layer	Description	Layer Depth (mm)	Rehab Date
L1	PMB Spray Seal	14	12-Mar-2013
L2	Primerseal	10	12-Mar-2013
L3	Bitumen Treated Base	150	12-Mar-2013
L4	Bitumen Spray Seal	10	22-Mar-2002
L5	Bitumen Spray Seal	10	22-Jun-1994
L6	Bitumen Spray Seal	10	10-Jul-1983
L7	Spray Seal - Quality Unknown	20	20-Sep-1965
L8	Granular - Quality Unknown	125	20-Sep-1965
L9	Subgrade - Quality Unknown	125	20-Sep-1965
L10	Subgrade - Quality Unknown	125	20-Sep-1965

6.12 OTHER ROADS

Other than the above mentioned roads, there were six other roads from which 58 pavement sections were included in the modelling. These roads were: Benaraby Rockhampton Road in Bruce Highway (Road ID: 10E), Rockhampton St Lawrence in Bruce Highway (Road ID: 10F), St Lawrence Mackay in Bruce Highway (Road ID: 10G), Mackay Proserpine in Bruce Highway (Road ID: 10H), Bowen Ayr in Bruce Highway (Road ID: 10K) and Ayr Townsville in Bruce Highway (Road ID: 10L). Detailed road locations are given in Table 6.14. As the trend of rutting and roughness progression in rehabilitated flood affected sections were similar to those roads discussed in sections 6.5 to 6.10, these roads were not analysed individually.

6.13 ANALYSIS OF SURFACE CONDITION OF RMS NSW ROADS

RMS NSW also provided surface condition data for flood affected roads, including roughness (NAASRA Roughness Meter -NRM roughness values in counts/km) and rutting data from 2009-2012. The road segment analysed here is a highway (Sturt Highway) constructed in 2008. This road was submerged for 10 days.

A marginal increase in roughness (NRM values), rutting (mm) and cracking (mm) was observed after flooding in some of the pavements which can be found in Figures 6.18 and 6.19.

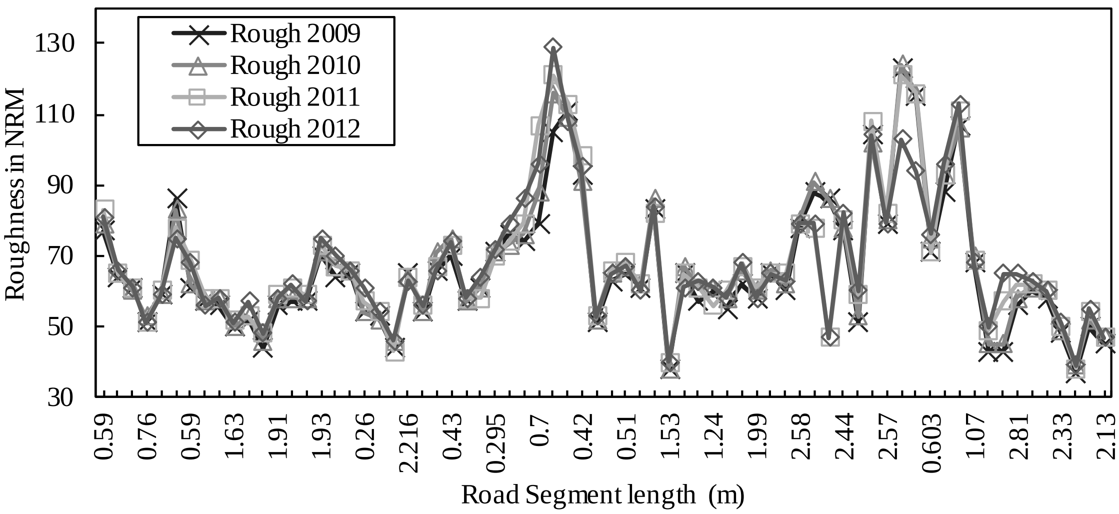


Figure 6.18: Comparison of pre- and post-flood roughness data of Sturt Highway, RMS NSW

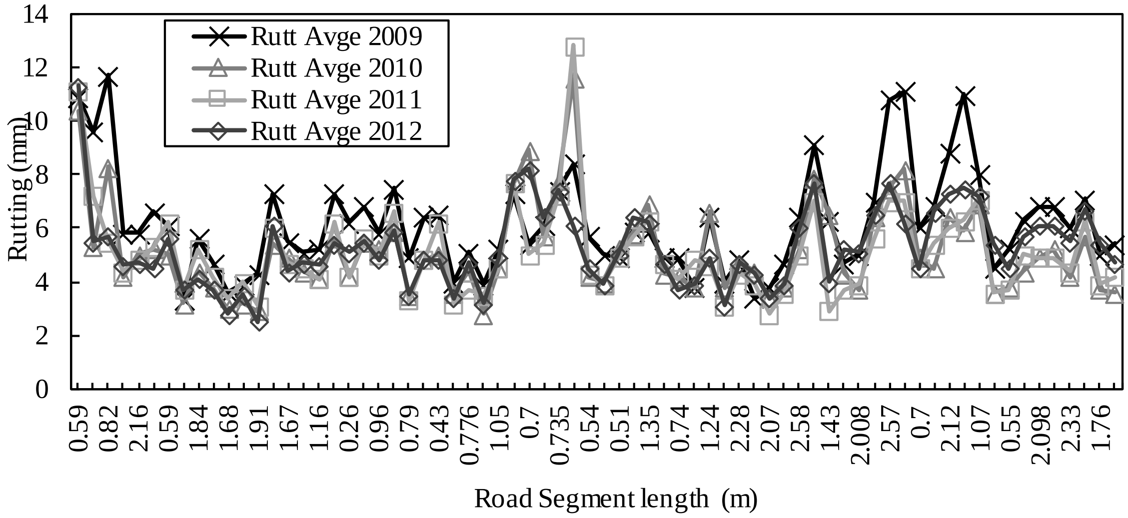


Figure 6.19: Comparison of pre- and post-flood rutting data of Sturt Highway, RMS NSW

6.14 TREND OF SURFACE CONDITION DETERIORATION

Some common trends were observed in the progression of pre- (2009 or 2010) and post-flood (2010 or 2011) rutting and roughness following the rapid deterioration phase. To understand pavement performance before and after a flood, data from some sections of flood affected roads were analysed and discussed in this section. Three common types of deterioration trends were observed for the rutting and roughness progression of flood affected pavement sections which are shown in Figures 6.20 to 6.23.

- Trend 1: refers to the sections that have post-flood rutting greater than pre-flood rutting but post-flood roughness less than pre-flood roughness (Figures 6.20 and

6.21, ID 42.6). This section was flooded again in 2013 resulting in the observed increase in rutting value.

- Trend 2: refers to the sections that have both post-flood rutting and roughness greater than pre-flood rutting and roughness (Figures 6.20 and 6.21, ID 39, 39.9 and 40.7). These sections were considered for modelling later in this chapter. Some of these sections were flooded again in 2013 or 2014 resulting in the observed increase in rutting values. This trend has two peaks in rutting values.
- Trend 3: also refers to sections that have both post-flood rutting and roughness greater than pre-flood rutting and roughness. However, the difference between trend 2 and trend 3 is that trend 3 clearly indicates that rutting and roughness progression of flood affected pavements depends on rehabilitation time and weather patterns or flooding occurrences in the area. Two sections of Rosewood Marburg Road in Figures 6.22 and 6.23 demonstrate how section rutting and roughness values improved after rehabilitation. There was no flooding event in the area after the 2011 flood and the pavement sections continue to perform well. In trend 2, pavement sections again started to deteriorate due to recurring floods.

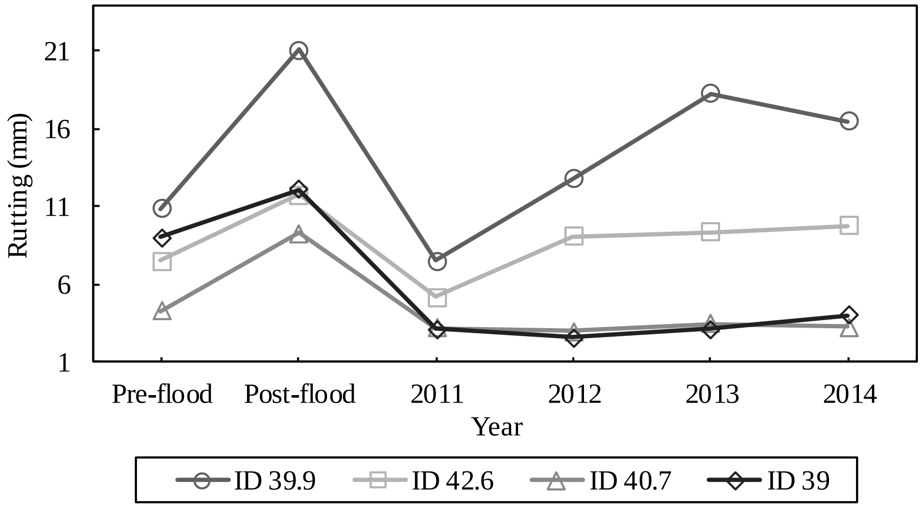


Figure 6.20: Rutting progression from 2009-2014 in Rockhampton Emu Park Road

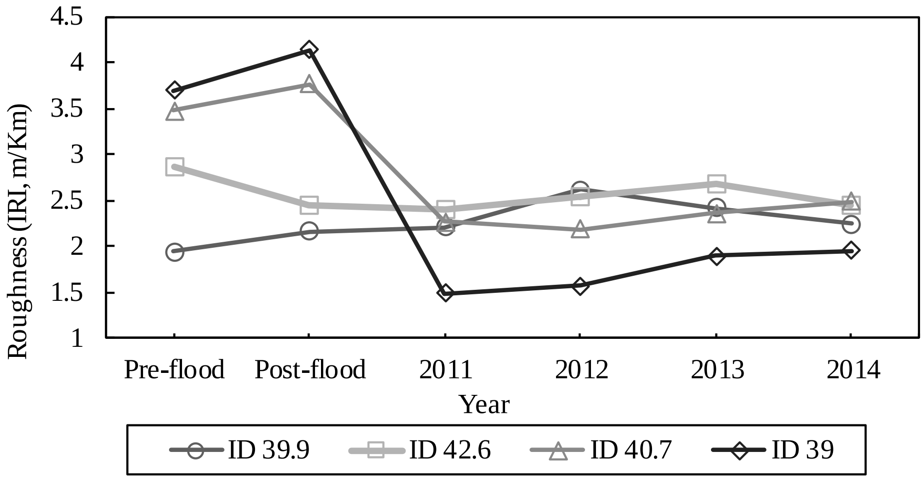


Figure 6.21: Roughness progression from 2009-2014 in Rockhampton Emu Park Road

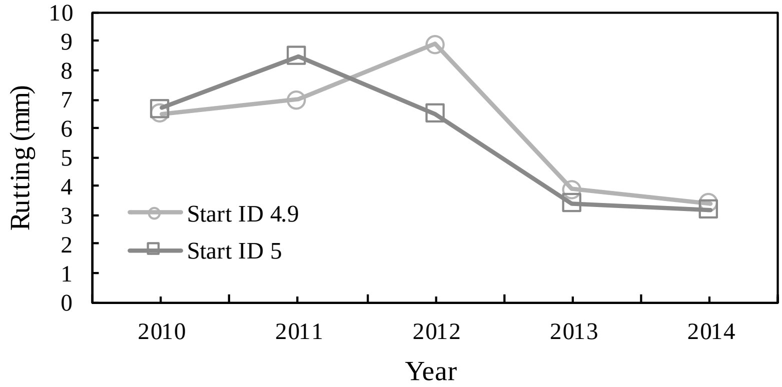


Figure 6.22: Rutting progression of Rosewood Marburg Road

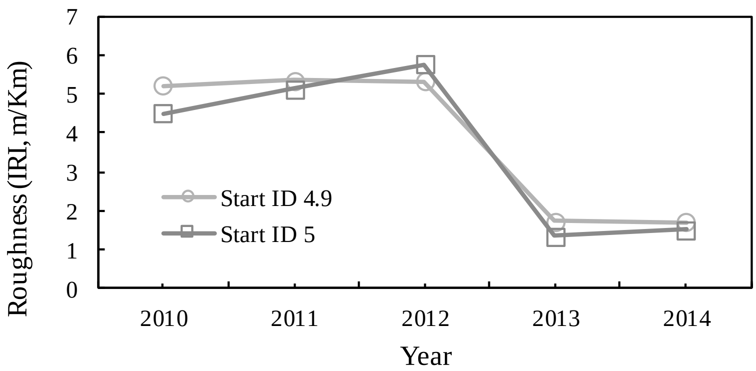


Figure 6.23: Roughness progression of Rosewood Marburg Road

6.15 DEVELOPMENT OF RUTTING AND ROUGHNESS MODEL (VARIABLE SELECTION)

This research developed two mechanistic-empirical, deterministic based deterioration models to predict the rapid deterioration of surface condition, such as rutting and roughness, of pavement impacted by river flooding, or the gradual rise of flood water. The observational data of local roads obtained from TMR, Queensland were used to develop the models. The data used in the modelling were collected following the flooding events from 2010-2013. As mentioned previously, the road sections included in modelling were rehabilitated as a result of the damage by the flooding event.

After the assessment of the flood affected pavements (discussed in sections 6.2 to 6.13), some pavement sections were selected for the development of the rutting and roughness models. This study considered a number of factors during selection of pavement sections and data processing. Different types of roads were used and the post-flood data were collected at different times following the flooding events in 2010 or 2011. The data for the deterministic models were prepared following the criteria outlined below:

- Sections that had pre- and post-flood rutting and roughness data (following the 2010 or 2011 flooding event), with post-flood values greater than pre-flood values, were selected to quantify the rapid deterioration phase after the flooding event.
- Sections with at least 5% loss of surface condition (rutting and roughness) were selected for the analysis to have significant increases in the deterioration of rutting and roughness. The term minimum loss refers to at least 5% increase in rutting and roughness post-flood, ΔIRI (change in IRI) is either greater than 0.1 or 5% increase in roughness.
- Both rutting and roughness models will be able to predict rutting and roughness within 172 days or approximately 25 weeks after flooding as the original data were collected within this timeframe. After 172 days, these models should not be used without calibrating the actual dataset and coefficients of the actual models.

In this study, the rate of increase in rutting following the flood was designated as $\Delta Rut_{post\text{-}flood}$ and the rate of increase in roughness following the flood was designated as $\Delta IRI_{post\text{-}flood}$. Both models were developed using nonlinear regression analysis.

For the rutting model, two independent variables were found to be statistically significant while for the roughness model, only one independent model was found to be statistically significant. The rutting model expresses increase in post-flood rutting ($\Delta Rut_{post\text{-}flood}$) as a function of time lapse in rutting measurement after flooding and the value of pre-flood rutting of individual pavement sections. The roughness model expresses increase in post-flood roughness ($\Delta IRI_{post\text{-}flood}$) as a function of time lapse in roughness measurement after flooding.

6.15.1 Rutting Model ($\Delta Rut_{post\text{-}flood}$)

Two forms of model for rutting were initially chosen which are given in Equations (6.3) and (6.4). The differences in pre- and post-flood rutting was designated as $\Delta Rut_{post\text{-}flood}$ which is $Rut_{post\text{-}flood}$ - $Rut_{pre\text{-}flood}$. The model expresses $\Delta Rut_{post\text{-}flood}$ as a function of time lapse in collection of rutting data after the flooding (t) and pre-flood rutting ($Rut_{pre\text{-}flood}$). After performing a number of trial and error runs of the non-linear regression analyses, Equation (6.5) was found to be the initial model for the selected data set with an R^2 value of 0.664. Tables 6.28 to 6.30 show the statistical analysis of Equation (6.5) and Tables 6.31 to 6.33 show the statistical analysis of Equation (6.6). Both multiple linear regression and non-linear regression analyses were undertaken to check the accuracy of the final derived model. After a number of trial and error runs in SPSS, Equation (6.5) was found to be the most appropriate for the selected data set with an R^2 value of 0.67.

$$\Delta Rut_{post\text{-}flood} = a \times time \wedge b + c \times Rut_{pre\text{-}flood} \tag{6.3}$$

$$\Delta Rut_{post\text{-}flood} = a \times time \wedge b + c \times Rut_{pre\text{-}flood} - d \tag{6.4}$$

$$\Delta Rut_{post\text{-}flood} = k_{rut} \times [0.081 \times time \wedge 0.849 + 0.03 \times Rut_{pre\text{-}flood}] \quad (R^2 = 0.664) \tag{6.5}$$

$$\Delta Rut_{post\text{-}flood} = k_{rut} \times [0.083 \times time \wedge 0.85 + 0.109 \times Rut_{pre\text{-}flood} - 0.746] \quad (R^2 = 0.67) \tag{6.6}$$

where,

$\Delta Rut_{post\text{-}flood}$	=	Differences between post-flood rutting and pre-flood rutting in mm (=$Rut_{post\text{-}flood}$- $Rut_{pre\text{-}flood}$)
$Rut_{pre\text{-}flood}$	=	Pre-flood rutting in mm
t	=	Time of collection of rutting data after flood in days (t < 172 days)
k_{rut}	=	local calibration factor for different types of pavement (default = 1)

The rutting model was developed with a coefficient of determination (R^2 value) of 0.67, with a 95% confidence interval and sample size, N = 436. The regression coefficients of the independent variables in Equation (6.6) are significantly correlated with the least standard error. Table 6.33 shows the t-values for the rutting model coefficients. All the coefficients are significant with $t > 2$ ($p < 0.005$).

Table 6.28: Parameter estimates for the rutting model

Parameter	**Estimate**	**Std. Error**	**95% Confidence Interval**	
			Lower Bound	**Upper Bound**
a	.081	.039	.004	.158
c	.030	.017	-.002	.063
b	.849	.095	.661	1.036

Table 6.29: Correlations of parameter estimates for the rutting model

	a	**c**	**b**
a	1.000	-.361	-.998
c	-.361	1.000	.324
b	-.998	.324	1.000

Table 6.30: ANOVA for the rutting model

Source	**Sum of Squares**	**Degrees of freedom**	**Mean Squares**
Regression	6273.608	3	2091.203
Residual	1731.152	433	3.998
Uncorrected Total	8004.760	436	
Corrected Total	5148.312	435	
Dependent variable: $\Delta Rut_{post\text{-}flood}$			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = .664.			

Table 6.31: Rutting model summary using stepwise linear regression

Model	**R**	**R Square**	**Adjusted R Square**	**Std. Error of the Estimate**
1	.819	.670	.669	1.98049
Predictors: (Constant), t ^ 0.85, $Rut_{pre\text{-}flood}$				

Table 6.32: ANOVA for the rutting model using stepwise linear regression

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3449.936	2	1724.968	439.780	.000[c]
	Residual	1698.376	433	3.922		
	Total	5148.312	435			
Dependent Variable: $\Delta Rut_{post\text{-}flood}$						
Predictors: (Constant), t ^ 0.85, Rut $_{pre\text{-}flood}$						

Table 6.33: t-values for the rutting model coefficients

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.746	.258		-2.891	.004
	t ^ 0.85	.083	.003	.819	29.618	.000
	$Rut_{pre\text{-}flood}$	.109	.031	.096	3.478	.001
a. Dependent Variable: $\Delta Rut_{post\text{-}flood}$						

Pre- and post-flood rutting plotted against time in a scatter plot to show the dataset used for modelling in presented in Figure 6.24. In Figure 6.25, $\Delta Rut_{post\text{-}flood}$ plotted against time to show the dataset used for modelling.

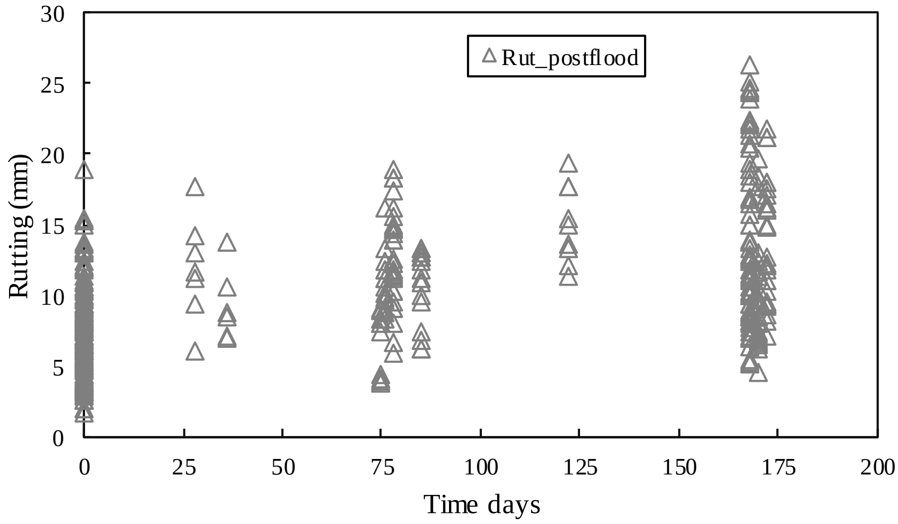


Figure 6.24: Pre- and post-flood rutting vs time

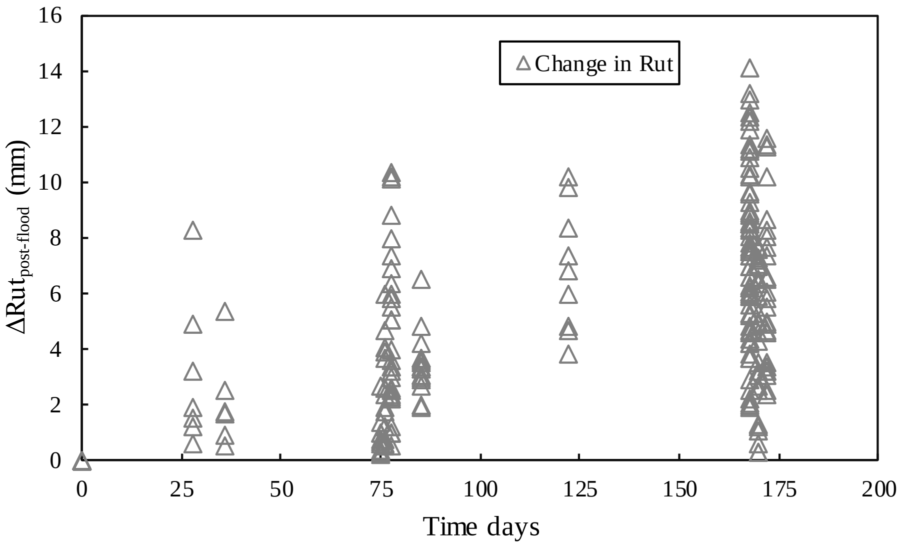


Figure 6.25: $\Delta Rut_{post\text{-}flood}$ vs time

6.15.2 Roughness Model ($\Delta IRI_{post\text{-}flood}$)

The differences in pre- and post-flood roughness were designated as $\Delta IRI_{post\text{-}flood}$ which is the $IRI_{post\text{-}flood}$ - $IRI_{pre\text{-}flood}$. The model expresses $\Delta IRI_{post\text{-}flood}$ as a function of time lapse in collection of roughness data after the flooding (t). The model was first predicted in the form of Equation (6.7). After performing a number of trial and error runs of the non-linear regression analyses, Equation (6.8) was found to be more appropriate for the selected data set. The statistical analyses of the model are presented in Tables 6.34 to 6.37.

The roughness model was developed with a coefficient of determination (R^2 value) of 0.319, sample size, N was 436 with a 95% confidence interval. The regression coefficients of the independent variables in Equation (6.8) have significantly good statistical correlation with the least standard error. Coefficients of all the parameters in the model (Table 6.37) also gives good results from t tests, with t greater than 2 ($t > 2$) and $p < 0.005$.

$$\Delta IRI_{post\text{-}flood} = a \times \sqrt{time} + b \quad (6.7)$$

$$\Delta IRI_{post\text{-}flood} = k_{rg} \times [0.039 + 0.027 \times \sqrt{time}] \; (R^2 = 0.319) \quad (6.8)$$

where,

$\Delta IRI_{post\text{-}flood}$	=	Differences between post-flood rutting and pre-flood rutting (=Roughness$_{post\text{-}flood}$ - Roughness$_{pre\text{-}flood}$) in IRI
t	=	Time of collection of roughness data after flood in days (t < 172 days)
k_{rg}	=	local calibration factor for different types of pavement (default = 1)

Table 6.34: Parameter Estimates for the Roughness model

Parameter	Estimate	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
a	.027	.002	.024	.031
b	.039	.015	.009	.069

Table 6.35: Correlations of Parameter Estimates for the Roughness model

	a	b
a	1.000	-.692
b	-.692	1.000

Table 6.36: ANOVA for the Roughness model

Source	Sum of Squares	df	Mean Squares
Regression	26.472	2	13.236
Residual	23.121	434	.053
Uncorrected Total	49.593	436	
Corrected Total	33.941	435	
Dependent variable: $\Delta IRI_{post\text{-}flood}$, R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = .319.			

Table 6.37: t value for the Coefficients in the Roughness model

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.039	.015		2.526	.012
	sqrt_timedays	.027	.002	.565	14.251	.000
a. Dependent Variable: $\Delta IRI_{post\text{-}flood}$						
b. Predictors in the Model: (Constant), √time						

Initially, the variable pre-flood rutting was also checked to see if there is any correlation with the increase in post-flood roughness. However, no significant correlation was found (t value for coefficient of Pre-flood rutting was found to be less than 2) and it was discarded from the model. Time is the only variable that impact the roughness model for rapid deterioration of flood affected pavements. In Figure 6.26, pre- and post-flood roughness are plotted against time to show the dataset used for modelling. In Figure 6.27, $\Delta IRI_{post\text{-}flood}$ is plotted against time to also show the dataset used for modelling.

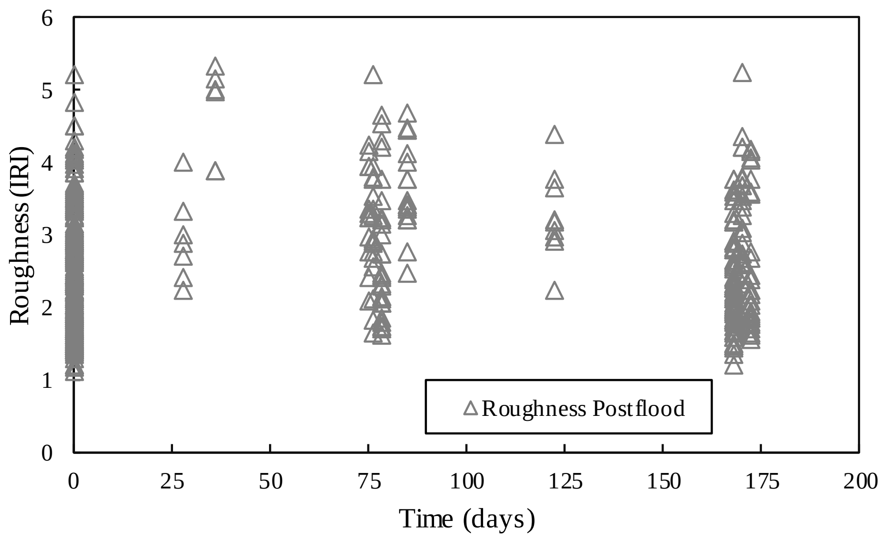


Figure 6.26: Pre- and post-flood roughness vs time

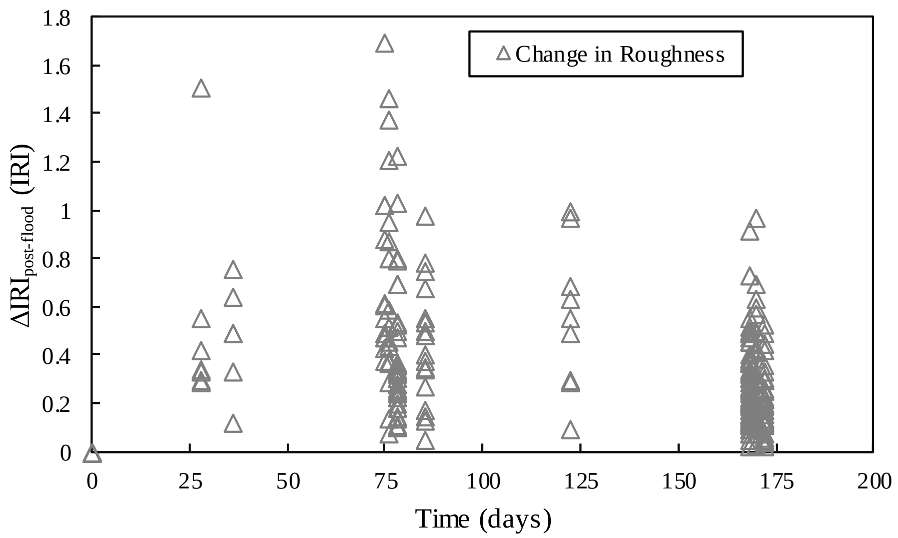


Figure 6.27: $\Delta IRI_{post\text{-}flood}$ vs time

6.16 GENERAL MEASURES/GUIDELINES TO BE FOLLOWED AFTER FLOODS

When quantifying the damage to pavements in flood-prone areas, some measures can be taken or guidelines can be followed which would facilitate damage analysis such as:

- Keeping record of intensity of flooding (mm)
- Height of flood water on the pavement
- Time of submergence under water during flood
- FWD testing immediately after flood water recedes
- Another FWD testing should be conducted several months after flooding
- How many potholes were created after flooding (loss of surface condition)
- Time to allow the heavy vehicle or debris hauling trucks on the flood affected road.

6.17 LONG-TERM MONITORING OF FLOOD AFFECTED ROADS

This section discusses a general overview and guidelines for strategic planning for investigation and long-term monitoring of flood affected roads. This strategic planning for investigation and long-term monitoring of the flood affected roads would improve the decision making process following any extreme weather event. Long-term monitoring of flood affected roads should be included in the decision making processes of the road agencies that are more at risk of frequent flooding. A schematic diagram in Figure 6.28 summarises the strategic planning. The first step is to identify the severely affected roads, after recent flooding, or to identify if the road had been built on a flood plain. A database should be built with the type of pavement material, surface layer, base, subbase or subgrade material, thickness of each layer, traffic category, any past or recent rehabilitation history, design CBR, and moisture content of the pavement. The roads can be categorised into those with similar traffic categories, pavement material and extent of flooding in similar geographic locations, which would reduce the cost of maintaining the database with a constrained budget. A good database will increase the accuracy of predicting pavement performance after flooding or extreme rainfall events. It will also reduce the cost of rehabilitation and maintenance.

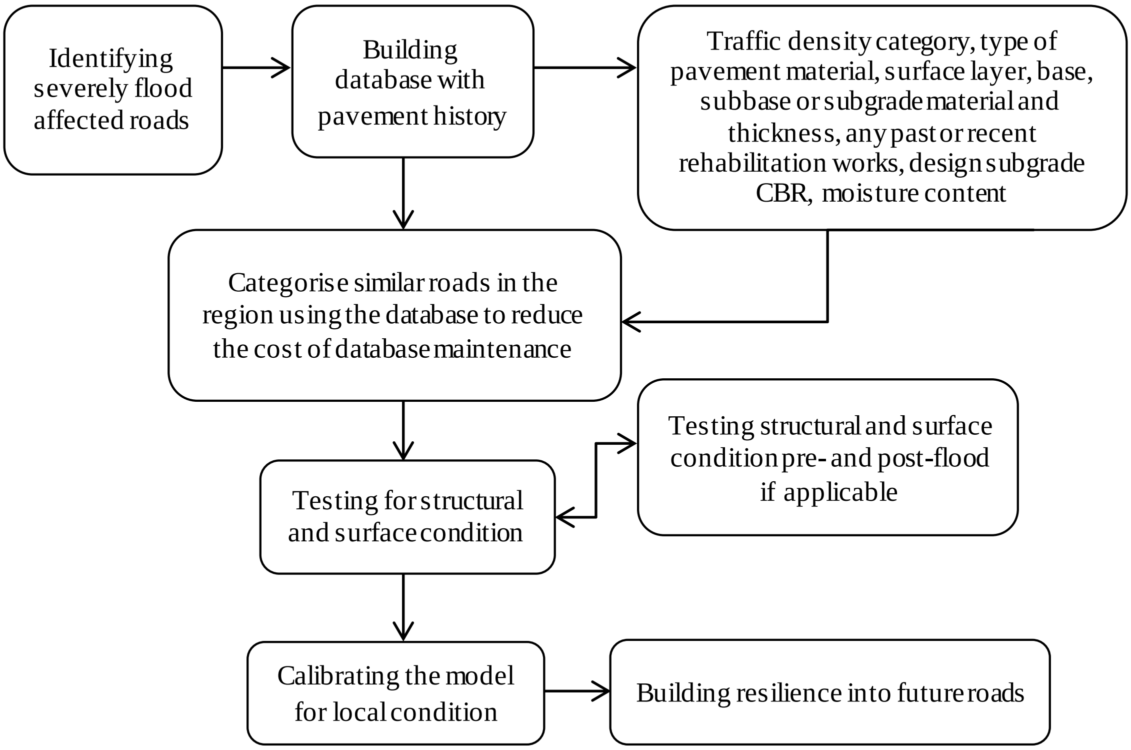


Figure 6.28: Strategic planning of long-term monitoring of the roads in flood affected areas

Testing the deflection is the best measure to check the structural strength immediately after a flood. A visual inspection can be done prior to measuring surface condition (roughness, rutting and cracking) to estimate the extent of deterioration. The rapid deterioration phase of the pavement after flooding or frequent extreme heavy rainfall events can be quantified by calibrating the model for local conditions. A calibrated model can improve decisions related to rehabilitation and building resilience into future roads, for example by using cement treated base for lightly trafficked local streets with thin AC (less than 60 mm) to building resilience against flood or extreme rainfall events.

6.18 SUMMARY

This chapter assessed the impact of floods on the surface condition of roads in Queensland. It presented two deterioration models for rutting and roughness that can be used by other road agencies after applying local calibration factors.

7. CONCLUSIONS

7.1 CONCLUSIONS

Roads were among the assets damaged when Queensland experienced its worst flooding in more than a century in January 2011. Many significant flooding events also occurred in Queensland and New South Wales between 2010-2013. This research assessed the impacts of floods on the structural and surface condition of pavements by comparing the pre- and post-flood data of partially or fully saturated pavements. The main outcomes of this research can be summarised as follows:

- Advanced knowledge and understanding of the impacts of extreme weather events, such as flooding, on pavements in Queensland and New South Wales.
- The development of four mechanistic-empirical deterministic based pavement deterioration models to predict the structural and surface condition (roughness and rutting) of pavements impacted by river flooding or gradual increase of flood water during the short term rapid deterioration phase.

Models developed for structural condition deterioration were based on the observational data of the flood affected pavements of the Brisbane City Council area. Models developed for rutting and roughness were based on the observational data of the flood affected pavements of Transport and Main Roads (TMR), Queensland. The models are sufficiently robust to be calibrated for the local road conditions. The accuracy in prediction of the post-flooding road network condition will reduce the cost of road maintenance for the state and local road agencies.

Models for Deterioration of Structural Condition

Pavement category, thickness of the asphalt layer and traffic density category were considered during the development of the models. Two models were developed using the same dataset to predict the rapid deterioration of post-flood structural condition.

The first structural deterioration model had one independent variable, time. The Modified Structural Strength ratio ($\mathrm{SNC}_{\mathrm{ratio}}$) was modelled as a function of time of collection of FWD deflection data after flooding (in days), and is given as:

$$SNC_{ratio} = 1.032 - 0.034 \times EXP^{(t/_{21.5})}$$

where,

SNC_{ratio}	=	Ratio of Modified Structural Strength of the pavement after time, t of flooding/ Modified Structural Strength before flooding, SNC_t/SNC_i
SNC_i	=	Modified Structural number before flooding
	=	$3.2 \times D_0(i)^{-0.63}$ (Paterson 1987)
$D_o(i)$	=	maximum deflection measured using a FWD before flooding
SNC_t	=	Modified structural strength or Modified structural number of the pavement after time, t of flooding
	=	$3.2 \times D_0(t)^{-0.63}$ (Paterson 1987)
$D_o(t)$	=	maximum deflection measured using a FWD after time, t of flooding
EXP	=	Exponential function
t	=	Time of collection of FWD data after flood in days (t < 42 days)

In the second structural deterioration model, the Modified Structural Number ratio for rapid deterioration phase of pavements after flooding was designated as SNC_{rapid}, which is the ratio of SNC_t/SNC_i. The model expresses the SNC_{rapid} as a function of time lapse in deflection measurement after flooding, subgrade strength (CBR) and MESA, and is given as:

$$SNC_{rapid} = k_f\,[1.227 - 0.312EXP\,(0.011t - 0.024\,(CBR + MESA))]$$

where,

SNC_{rapid}	=	Ratio of the post-flood SNC value (at time t) divided by the SNC value pre-flood, SNC_t/SNC_i
y_0	=	Constant
b, c	=	Coefficients
SNC_i	=	Modified Structural number before flooding
	=	$3.2 \times D_0(i)^{-0.63}$ (Paterson 1987)
$D_o(i)$	=	maximum deflection measured using a FWD before flooding
SNC_t	=	Modified structural strength or Modified structural number of the pavement after time, t of flooding
	=	$3.2 \times D_0(t)^{-0.63}$ (Paterson 1987)

$D_o(t)$	=	maximum deflection measured using a FWD after time, t of flooding
EXP	=	Exponential function
t	=	Time of collection of FWD data after flood in days (t < 42 days)
CBR	=	Subgrade strength
MESA	=	Millions of equivalent standard axles over 20-year design life
k_f	=	local calibration factor for different types of pavement (default=1)

This three parameter model is dependent on the local conditions and pavement types. It can predict the deterioration of pavements within 42 days, i.e. within six weeks of flooding. This equation can be used to estimate the post-flood SNC if the pre-flood, SNC is known. After six weeks, this model becomes invalid and may not be applicable.

Models for Rutting and Roughness

This research developed two mechanistic-empirical deterministic based deterioration models to predict the rapid deterioration of surface condition, rutting and roughness, of the pavement impacted by river flooding or the gradual rise of flood water. The observational data of local roads obtained from TMR, Queensland were used to develop the models.

The difference in pre- and post-flood rutting was designated as $\Delta Rut_{post\text{-}flood}$ which is the $Rut_{post\text{-}flood}$ - $Rut_{pre\text{-}flood}$. The model expresses $\Delta Rut_{post\text{-}flood}$ as a function of time lapse in collection of rutting data after the flooding (t) and pre-flood rutting ($Rut_{pre\text{-}flood}$), and is given as:

$$\Delta Rut_{post\text{-}flood} = k_{rut} \times [0.083 \times time \wedge 0.85 + 0.109 \times Rut_{pre\text{-}flood} - 0.746]$$

where,

$\Delta Rut_{post\text{-}flood}$	=	Differences between post-flood rutting and pre-flood rutting in mm (= $Rut_{post\text{-}flood}$- $Rut_{pre\text{-}flood}$)
$Rut_{pre\text{-}flood}$	=	Pre-flood rutting
t	=	Time of collection of rutting data after flood in days (t < 172 days)
k_{rut}	=	local calibration factor for different types of pavement (default = 1)

The difference in pre- and post-flood roughness was designated as $\Delta IRI_{post\text{-}flood}$ which is the $Roughness_{post\text{-}flood}$ - $Roughness_{pre\text{-}flood}$. The model expresses $\Delta IRI_{post\text{-}flood}$ as a function of time lapse in collection of roughness data after the flooding (t), and is given as:

$$\Delta IRI_{post\text{-}flood} = k_{rg} \times [0.039 + 0.027 \times \sqrt{time}] \quad (R^2 = 0.319)$$

where,

$\Delta IRI_{post\text{-}flood}$	=	Differences between post-flood roughness and pre-flood roughness in IRI (= $Roughness_{post\text{-}flood}$ - $Roughness_{pre\text{-}flood}$)
t	=	Time of collection of roughness data after flood in days (t < 172 days)
k_{rg}	=	local calibration factor for different types of pavement (default = 1)

Limitations of use for the Models

The structural deterioration models could predict the pavement performance within six weeks of flood as the original data set was collected within this timeframe. Rutting and roughness models could predict rutting and roughness within 172 days or approximately 25 weeks of flooding as the original data set was collected within this timeframe. After 172 days, these models may not be applicable. Calibrating the models for the local conditions would allow pavement engineers to quantify the loss of structural strength and surface condition (rutting and roughness) post-flood. The deterioration models can predict the immediate impact of flooding on the pavement. By quantifying the deterioration from the model, local road authorities can make more effective decisions about whether pavement rehabilitation or resurfacing is required.

Other Outcomes

The following conclusions were made from a detailed assessments of the flood affected pavements:

- It was found that flood affected pavements tend to regain pre-flood structural strength, either fully or partially, after rehabilitation followed by dry weather in the area. However, some pavements did not return to their pre-flood strength / surface condition, even after rehabilitation and needed frequent repair or rehabilitation following the initial flooding event or due to recurring flooding events. The failure of some pavement sections to fully regain their pre-flood strength suggests that road agencies will face increasing demand for rehabilitation in the future due to the decreasing strength of pavements compared to their original deterioration predictions. Hence, frequent occurrences of extreme weather events, such as flooding, will increase the cost of road maintenance.

- Flood affected pavements were found to be deteriorated rapidly rather than gradually as anticipated by many available deterioration models. This short-term phase was designated as the rapid deterioration phase in this research.
- The assessments demonstrated that AC pavements with CTB performed significantly better than Thin AC pavements with gravel base after the January 2011 flood in the observational data of the Brisbane City Council area. This information can be used for building future resilience in pavements in flood prone areas.
- The assessments further indicated that pavements with low pre-flood rutting had significantly lower post-flood rutting than pavements with high pre-flood rutting in the TMR observational data. This indicates that pavements with low pre-flood rutting are more likely to survive well after the flooding while a pavement with high pre-flood rutting (pavement in poor condition) is more likely to deteriorate following the flooding event.

Finally, a presentation has been made outlining a general overview and guidelines for strategic planning for investigation and long-term monitoring of flood affected roads. It would improve the decision making process following any extreme weather event. Results suggest that long-term monitoring of flood affected roads should be included in the decision making processes of road agencies that are at more risk of frequent flooding.

The innovation of this thesis is the development of the models for flood-affected pavements. After calibrating for the local conditions, these models can be used in the pavement management system (PMS) of the local road agencies to predict deterioration of flood affected pavements which was not done by any other study previously. Moreover, the methodology followed in this study can be replicated by local road agencies in other countries that have flooding issues, to predict the deterioration of flood-affected pavements and to improve the efficiency of the PMS.

7.2 RECOMMENDATIONS FOR FURTHER STUDIES

Road agencies either need to build flood resilient roads in flood prone areas or carefully maintain flood affected roads if building resilience in existing roads is not an economical option. The long-term observation of flood affected pavements is recommended to predict their deterioration, with any degree of confidence, compared to deterioration of non-flooded pavements.

If the saturated road is loaded immediately after flooding, the impacts of flooding can be exacerbated. However, when answering the question of how long the road should be closed immediately after a flood, the impact on road users also needs to be considered. It may not always be possible to keep the road closed to traffic for a long time due to socio-economic reasons or due to the impact of road closures on the local residents. If a road must be opened after the flood water recedes and heavy traffic permitted for emergency use, the models and investigation program described in this research cannot predict the duration of road closure but can help to estimate the extent of damage and deterioration to pavements during the rapid deterioration phase immediately after the flood. Local engineers can be ready for road rehabilitation without delay. However, future studies should include the topic of road closure in the decision making process.

This study clearly indicated that subgrade strength of flood affected pavements was affected by the major flooding event in January 2011. Pavements needed urgent repair and maintenance. It was vital that they were treated as soon as possible because the decreasing quality and serviceability of these pavements would be more rapid than normal rates without rehabilitation. It is highly recommended that the use of CTB or stabilized base for pavements in flood-affected areas should be included in future studies.

In future, a full scale accelerated load test could be done by artificially flooding a road section to simulate a gradual rise of flood water. The structural strength and surface condition of the pavement should be measured regularly along with moisture content variation to check the effect of moisture on pavements. In this study, due to the lack of pre- and post-flood moisture content data, it was impossible to include this parameter in the models. However, it is an important parameter and should be studied in future.

This study did not include analysis of the structural strength of TMR, Queensland pavement sections due to the lack of deflection data before and after the recent flooding events. However, to enable this issue to be addressed by future research, the appropriate data should be collected following any extreme weather events, such as flooding. Then, this data can be used to analyse and model the deterioration of flood affected roads over the long term.

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APPENDICES

APPENDIX A

Terms related to Probabilistic Models of Khan et al. (2014b)

Markov chain: 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.

TPM: The Markov chain may or may not be time dependent. A time-independent Markov chain is known as a homogeneous TPM. When the transition probability is assumed to change with time, then a non-homogeneous Markov chain results; this is called a non-homogeneous TPM. Non-homogeneous TPMs represent real Road deterioration (Khan et al. 2014b).

APPENDIX B

Table: Road Name: Warrego Highway, Road Section Name: Miles Roma, Road Section ID: 18D

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	22.3	22.4	10.4	12.4	14.6	5.3	5.9	2.7	3.38	2.94	1.26	1.69
18D	22.4	22.5	9.8	11.8	12.1	6.7	7.8	2.92	3.19	3.2	1.3	1.92
18D	22.5	22.6	6.4	9.5	9.6	7.2	2.7	3	3.4	3.78	1.79	1.84
18D	22.6	22.7	9.7	13	12.4	6.6		4.08	4.43	4.16	2.06	2.32
18D	22.7	22.8	9.9	11.7	13.4	7.1	7.5	3.55	3.54	3.49	1.54	2.1
18D	22.8	22.9	9.3	11.2	13	7.1	7	3.23	3.77	4.2	1.22	2.07
18D	22.9	23	9.3	12.7	12.1	7.1	8	3	3.75	4	1.58	2.25
18D	24	24.1	7.4	10.8	12	5.5	6.1	3.96	4.46	4.34	1.52	2.39
18D	24.1	24.2	6.2	10.5	11.3	5.8	6.6	4.53	4.2	3.97	1.32	2.04
18D	24.6	24.7	9.9	12.9	13.3	3.9	4.9	2.7	2.75	2.85	1.08	1.39
18D	25.4	25.5	8.4	13.2	3.2	4.8	6	3.14	4.12	1.72	2.16	3.07
18D	25.5	25.6	8.5	12.7	3.2	4.5	5.3	3.9	4.68	2.18	2.14	2.61
18D	25.6	25.7	10.2	13.3	4.3	7.6	8.5	4.3	4.43	1.68	2.1	2.83
18D	26.7	26.8	6.3	9		6.7	9	2.85	2.47		1.62	1.61
18D	27.7	27.8	8.4	11.4		3.6	3.5	2.45	2.39		2.16	3.86
18D	27.8	27.9	12	15.5	7.9	3.9	3.4	2.31	2.23		1.67	2.07
18D	29.7	29.8	4.8	7.1	7.5	3.1	2.8	4.18	3.67	3.79	1.45	1.68
18D	30.3	30.4	6.9	9.1	11	3.2	3.4	4.71	4.63	4.84	1.47	1.88
18D	30.8	30.9	9.9	12.6	13.4	0.2	4.7	3.62	4	4.02	2.14	2.48
18D	31	31.1	9.3	11.8	14.5	3.4	5	4.12	3.94	3.98	1.36	2.05

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	31.6	31.7	7.1	10.4	10.7	4.3	4.5	3.31	3.32	4.41	1.49	2.32
18D	32.3	32.4	8	14.2	14.5	4.2	5.5	3.14	3.16	3.41	1.88	2.27
18D	34.9	35	6.4	9.9	7.8	2.7	4.5	2.98	3.46	4.12	1.51	2.08
18D	35.2	35.3	6.9	8.6	9.6	5.1	6.4	4.34	3.96	4.02	2.12	2.58
18D	35.9	36	5.8	7.9	8.4	7.9	8.1	2.55	2.42	2.39	2.45	2.32
18D	36	36.1	5.3	9.2	8	12	7.7	2.84	2.84	2.98	2.82	3.17
18D	36.1	36.2	7.6	10.5	10.6	7.4	3.7	3.18	3.2	2.76	2.14	3.14
18D	58.9	59	6.1	8.4	5.2	5.3	4.6	3.54	3.25	2.03	2.05	3.23
18D	59	59.1	6.3	9.5	5.4	6.8	5.5	3.48	3.42	2.05	2.01	2.44
18D	59.7	59.8	7.8	9.8	6.5	3.4	2.4	3.6	3.64	1.61	1.87	3.55
18D	60.1	60.2	5.7	9.2	5.5	4.9	5.5	4.38	3.64	1.14	1.24	2.29
18D	65.5	65.6	6.1	12.6	9.5	7.7	13.6	2.87	3.42	2.13	2.56	2.91
18D	88.6	88.7	9	11.7		13.9	14.2	1.83	1.46		1.78	1.76
18D	88.8	88.9	3.9	6.1		6.4	5.3	2.52	2.21		1.75	1.67
18D	91	91.1	3.5	6.1	7.2	4.3	9.1	2.8	2.63	2.89	2.61	2.77
18D	91.1	91.2	3.1	6.7	6.6	2.4	2.4	2.72	3.2	2.6	2.69	2.62
18D	91.2	91.3	2.5	6.2	5.4	6.1	9.1	2.31	2.46	3.23	3.18	3.14
18D	91.3	91.4	3.6	7.3	7.8	7.3	7.1	3.02	3.36	3.72	3.93	4.25
18D	97.7	97.8	3.2	6.1	6	5.7	7	3.1	3.27	4.08	1.33	1.41
18D	101.1	101.2	6.9	8.9	8.8	3.6	5	4.3	3.12	4.22	1.73	1.6
18D	102.8	102.9	5.7	9.4	9	2.5	1.6	4.1	4.31	4.5	1.2	1.58
18D	102.9	103	6.4	8.6	10.1	2.5	2.7	4.1	4.09	4.6	1.24	2.29
18D	103.1	103.2	5.6	7.8	8.3	3.3	3.9	4.53	4.74	4.89	1.24	1.67
18D	103.3	103.4	3.7	5.9	5.6	2.4	2.7	4.43	4.43	4.38	1.13	1.76

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	103.4	103.5	5.4	9.9	8.1	3.2	3.1	4.15	4.16	4.38	1.08	1.47
18D	103.5	103.6	5.6	7.8	6.4	2.7	3.6	4.03	3.43	3.97	0.98	1.26
18D	104.7	104.8	8.4	10.7	9.1	3	2.9	3.95	4.21	4.29	0.9	1.34
18D	107.4	107.5	12.7	14.9	13.8	4.5	6.1	4.07	3.94	4.52	1.49	2.57
18D	110.7	110.8	3.5	7.4	1.7	2	2.2	1.74	1.79	1.57	1.26	1.43
18D	112.3	112.4	4	6.2	4.8	5.2	4.6	1.64	1.78	1.18	1.14	1.25
18D	116.6	116.7	3.7	5.9	1.7	2.8	3.5	3.19	3.5	1.12	1.12	1.37
18D	116.7	116.8	3.9	6.6	1.5	2.7	3	3.11	3.36	1.19	1.59	2.33
18D	116.8	116.9	3.7	8	1.5	1.7	2.6	3.54	3.06	1.12	1.25	1.65
18D	123.1	123.2	8.4	10.6	12.1	12.4	10	1.84	2.28	2.62	2.67	2.71
18D	123.7	123.8	3.1	6	9.3	8.3	9	1.77	1.48	1.81	1.45	1.52
18D	123.8	123.9	9.1	14.5	15.2	7	4	2.5	2.44	2.82	1.99	2.64
18D	126.7	126.8	4.1	7	5.8	10.8	10.5	2.43	2.69	2.54	3.18	3
18D	126.8	126.9	8	12.6	9.8	11.2	10.3	3.82	2.76	3.5	2.95	3.05
18D	131.4	131.5	4.2	6.4	1.7	2.3	4	3.06	2.88	1.29	1.38	1.48
18D	131.7	131.8	3.3	5.7	1.6	2.5	2.9	2.41	2.08	1.88	2.08	2.09
18D	131.8	131.9	3.6	5.6	1.4	2.6	2.4	2.75	2.72	1.84	2.11	2.19
18D	132.1	132.2	3.4	6.2	2.1	4.2	5.2	2.16	2.14	1.05	1.58	1.31
18D	132.2	132.3	4.6	7.5	1.9	4.5	3.5	2.31	2.18	1.46	1.75	1.87
18D	134.7	134.8	7.3	12.2		5	7.3	2.27	2.16		1.63	1.84
18D	134.8	134.9	9.6	14.2		3.8	5.3	1.98	1.94		1.57	1.55
18D	134.9	135	8.8	17		4.4	7.4	2.33	2.14		1.86	1.93

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	135	135.1	10.4	15		-0.1	8.6	2.44	2.82		1.62	1.65
18D	135.1	135.2	10	13.8	2.3	5.7	7.9	1.84	2.24	2.86	3.15	2.86
18D	21	21.1	5	5.1	5.4	5.6	4.7	2.08	1.98	1.93	2.35	2.45
18D	21.1	21.2	5.9	6.5	6.7	7	8.9	2.04	1.99	2.07	2.16	2.35
18D	21.3	21.4	8.7	9.4	9.7	4.2	7.5	2.96	2.95	3.55	1.66	2.99
18D	21.4	21.5	6.1	6.3	6.1	5.2	8.1	3.08	3.16	3.27	1.87	1.8
18D	23.2	23.3	4.9	5.4	5.3	6.3	4.4	2.18	2.01	1.69	1.17	1.4
18D	23.5	23.6	3.9	5.4	7.4	5	4	3.13	2.72	3.14	1.49	1.83
18D	23.6	23.7	8.3	9.8	10.5	5.4	5.3	4.13	3.88	4.12	1.72	2.09
18D	23.7	23.8	7	7.3	7.4	4.8	5.5	3.51	3.5	3.79	1.39	2.62
18D	23.9	24	8	9.2	10.5	6.3	8.1	3.2	2.74	3.17	1.55	1.73
18D	24.7	24.8	5	5.1	5.2	4.9	6.9	1.98	1.78	2.09	0.97	0.98
18D	25.1	25.2	8.9	9	7.6	4.6	6.4	2.63	2.55	2.87	1.55	1.75
18D	25.2	25.3	8.1	9	5.2	5.5	4.8	2.69	2.59	1.51	1.44	1.63
18D	25.3	25.4	7.8	8.9	4.8	7	5.8	3.53	3.04	1.84	1.78	2.79
18D	25.8	25.9	6.6	8.1	3	4.5	4.6	2.8	2.79	1.66	1.7	2.33
18D	25.9	26	6.4	8.2	2.8	3.6	5.2	2.54	2.49	1.99	1.79	1.76
18D	26	26.1	6.8	7.6		3.3	6.8	2.41	2.31		1.69	2.2
18D	26.4	26.5	6.9	7.3		3.6	3.4	3	3		1.6	1.71
18D	26.5	26.6	5.2	6		3.7	4.8	2.93	2.64		1.51	2.3
18D	26.6	26.7	3.8	4		5	7.5	3.04	2.89		1.72	2.14
18D	26.8	26.9	8.8	10.1		7	9.8	2.8	2.79		1.69	2.37
18D	27.2	27.3	4.6	5.1		0	6.6	2.08	1.83		1.83	1.85
18D	27.6	27.7	6.9	7.9		4.3	2.8	2.35	2.34		1.92	1.54

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	27.9	28	13.2	13.2	11.2	3.8	5.4	2.1	1.87	2.1	1.73	1.17
18D	28	28.1	10.6	12	9.9	3.6	3.2	2.41	2.18	2.34	1.99	2.25
18D	28.1	28.2	11	12.2	10.7	3.8	4	2.36	2.51	2.35	2.37	2.27
18D	28.2	28.3	7.3	7.3	5.4	3.6	4.5	2.22	2.15	2.25	2.26	2.38
18D	29.2	29.3	7.4	9	8.9	3.1	6.7	3.84	3.68	3.8	2.05	3.1
18D	29.3	29.4	6	7.3	6.8	3.1	4.1	4.3	3.88	4.56	2.28	3.5
18D	29.4	29.5	6.9	8.4	9.7	0	7.7	3.84	3.1	3.3	2.02	2.79
18D	29.5	29.6	7.1	7.2	7.6	3.3	4.8	3.87	3.5	3.81	1.97	2.59
18D	29.6	29.7	5.5	6.9	6.1	3.9	3.8	4.37	4.23	4.43	1.91	2.74
18D	30.1	30.2	10	11.1	14.1	3.5	4.5	3.88	3.46	3.71	1.75	2.81
18D	30.3	30.4	6.9	9.1	11	3.2	3.4	4.71	4.63	4.84	1.47	1.88
18D	30.4	30.5	11.3	12.7	14.7	3.1	3.1	4.96	4.7	5.05	1.7	2.37
18D	32.6	32.7	5.2	6.8	6.6	5	5.2	2.39	2.79	2.87	1.8	1.78
18D	32.8	32.9	10.4	12.2	11.8	2.7	7.1	2.5	2.66	3.33	1.59	1.45
18D	32.9	33	8.2	9.4	11.7	3.8	7.1	2.07	2.39	2.48	1.8	1.81
18D	33	33.1	10.6	10.8	12.7	0.5	7.4	2.2	2.56	2.43	1.59	1.88
18D	33.3	33.4	7.9	9.3	11	2.9	6.2	3	2.86	3	1.56	2.01
18D	33.5	33.6	4.1	4.8	5.5	3.6	4.2	3.33	2.78	3.24	1.59	3.28
18D	33.6	33.7	3.4	3.9	4.6	3.3	4.2	2.67	2.56	2.57	1.65	2.21
18D	34.2	34.3	4	4.6	4.3	4.1	14.7	2.82	2.29	2.99	2.23	2.9
18D	34.7	34.8	6.4	8.8	10.8	3	4	4.27	3.89	4.45	1.56	3.99
18D	36.3	36.4	5.3	6.9	4.7	3.1	3.1	3.54	3.43	3.23	2.83	3.53
18D	37.2	37.3	6.4	6.5	5.8	1.3	4.8	2.65	2.84	3.01	2.77	2.66
18D	52.4	52.5	3.2	3.9	3.9	4	4.9	1.5	1.61	1.7	1.54	1.94

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	52.8	52.9	3.3	3.7	4.7	4.4	4.1	1.31	1.25	1.43	1.44	1.45
18D	52.9	53	3.5	4	4.5	4	4.8	1.41	1.46	1.59	1.47	1.47
18D	53.2	53.3	3.1	3.3	4.1	4.7	4.9	1.63	1.61	1.65	1.81	1.79
18D	53.3	53.4	3.5	4.3	3.7	4.5	3.8	1.58	1.61	1.64	1.7	1.81
18D	53.7	53.8	3.6	3.7	4	4.3	4.5	2.36	2.3	2.29	2.18	2.51
18D	54.1	54.2	3.6	3.7	3.5	3.9	4.3	2.2	1.92	2.13	2.13	2.84
18D	54.5	54.6	5.3	5.8	2.4	3	2.7	3.53	3.07	2.19	2.29	2.62
18D	54.6	54.7	4.8	5.2	1.3	2	2.8	3.22	3.04	1.6	1.78	2.4
18D	54.7	54.8	3.3	4.3	1.2	2.7	3.7	2.71	2.61	2.02	2.06	2
18D	54.8	54.9	3.8	4.2	3.6	4.3	5.9	2.71	2.48	2.9	2.81	3.16
18D	55	55.1	3.7	3.9	5	5.8	6.5	2.06	2.29	2.23	2.19	2.52
18D	55.1	55.2	4.1	4.6	4.9	4.7	4.1	2.11	2.18	2.15	2.17	2.05
18D	55.2	55.3	5.9	6.7	7.7	8.8	9.2	2.47	2.24	2.64	2.51	2.37
18D	55.3	55.4	7.1	8.5	7.1	8.8	6.4	2.55	2.77	2.92	2.68	2.67
18D	55.4	55.5	4.5	5.5	3.5	4.8	3.4	3.17	3.15	2.56	2.44	2.6
18D	55.5	55.6	5.4	5.9	1.9	4.4	2.4	3.05	2.79	2.32	2.17	2.37
18D	55.6	55.7	5.7	6.1	3.7	6.7	3	2.27	2.03	1.87	2.01	2.01
18D	57	57.1	5.3	5.6	5.4	6.6	5	3.79	3.58	1.93	1.68	2.35
18D	57.1	57.2	5.7	5.6	8.3	7.3	5.4	3.57	3.14	1.51	1.39	1.47
18D	57.2	57.3	7.5	7.3	9.1	8.3	9.3	3.07	2.48	1.55	1.54	1.64
18D	57.3	57.4	10.2	10	7.8	7.3	6	3.57	3.28	1.55	1.6	2.06
18D	57.4	57.5	4.3	4.4	6.9	6.9	6	2.33	2.3	1.37	1.4	1.64
18D	57.5	57.6	5.2	5.3	5.4	5.7	3.5	3.2	2.79	1.22	1.35	1.69
18D	57.6	57.7	8.1	6.4	4.4	4.6	4.2	2.79	2.3	1.5	1.64	2.14

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	57.7	57.8	4	4.5	5.5	4.9	7.9	2.74	2.38	1.59	1.78	2.35
18D	57.8	57.9	6	7.3	4.9	4.1	5.1	3.07	3.22	1.89	1.97	2.38
18D	57.9	58	5.2	7.9	4.7	4.3	4.9	3.53	3.46	1.8	2.21	2.46
18D	58	58.1	7.5	6.8	5	5	6.4	3.37	3.46	1.76	1.92	2.95
18D	58.1	58.2	5.5	5.5	4.3	4.3	4.1	2.8	2.28	1.5	1.76	1.85
18D	58.2	58.3	5.7	5.8	4.1	4.7	5.2	2.75	2.75	2.18	2.15	2.33
18D	58.3	58.4	4.5	5.1	3.7	4.7	4.2	2.67	2.51	2.58	2.61	2.58
18D	58.4	58.5	6.5	6.9	3.7	4.5	3.9	3.46	3.14	1.97	2.33	2.65
18D	58.5	58.6	6.7	7.1	4.2	5.4	6.5	3.05	3.13	1.9	2.3	2.45
18D	58.7	58.8	6.2	6.4	5.2	3.8	4.1	4.1	4.16	2.13	2.34	2.49
18D	58.8	58.9	7.3	7.6	5	4.6	6.5	3.24	3.05	1.8	2.03	1.98
18D	59.1	59.2	5.9	7	5.9	8.4	7.9	3.82	3.58	2.21	2.05	2.19
18D	59.3	59.4	5.2	6.2	5.5	6.8	7.1	3.53	3.28	1.66	1.62	1.74
18D	59.4	59.5	4.9	5.8	5	7	6.2	3.27	3.08	1.6	1.74	2.59
18D	59.5	59.6	4	4.1	7.4	6.4	8.8	3.03	3.19	1.84	1.89	2.1
18D	59.6	59.7	6	6	10.3	5.2	5.6	3.82	3.91	1.8	2.04	2.69
18D	59.8	59.9	7.1	7.7	4	3.1	5.8	3.7	3.35	1.57	1.86	2.43
18D	60.3	60.4	4.3	4.9	5.8	6.1	5.7	2.71	2.48	1.68	2.19	3.62
18D	60.4	60.5	5.8	6.1	7.1	6.3	6.8	2.95	2.85	1.51	1.9	2.89
18D	60.9	61	10.6	12.3	6.7	5.2	9.1	3.84	3.72	1.55	1.74	2.33
18D	61.5	61.6	4.1	4.2	5.3	4.4	6.5	2.71	2.6	2.6	2.78	3.13
18D	61.8	61.9	3.7	3.8	3.6	4	4.3	2.65	2.47	2.56	2.58	2.94
18D	62	62.1	3.4	4.3	5.8	5.4	5.2	1.87	2.09	2.18	2.28	2.1
18D	62.5	62.6	8.1	10.1	11.8	10.2	9.6	3.13	3.25	3.46	3.43	2.83

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	62.6	62.7	9.9	12.5	11.3	9.9	11.5	3.27	3.33	3.27	3.54	4.07
18D	62.7	62.8	6.2	6.8	9.7	8.3	11.5	2.67	2.87	2.96	3.1	2.81
18D	62.9	63	5.4	5.7	5.1	5.9	5	2.93	2.89	3.06	3.07	3.05
18D	63.2	63.3	3.8	4.3	4.9	3.8	5.2	2.07	1.78	1.67	1.89	2.07
18D	63.3	63.4	7	7.3	7.7	6.5	7.8	2.02	1.93	2.14	2.05	1.99
18D	63.6	63.7	3.3	3.6	4.1	4.7	3.9	1.33	1.34	1.42	1.57	1.33
18D	64.2	64.3	3.4	3.5	4.4	4.7	5.7	2.77	2.82	2.96	2.83	2.88
18D	64.5	64.6	3.5	3.5	3.5	3.3	3.9	2.57	2.66	3.04	2.92	2.54
18D	64.7	64.8	3.3	3.3	3.3	3	3.4	3.02	3.07	3.37	3.16	2.99
18D	64.8	64.9	4.8	4.8	3.3	3.3	3.2	3.7	3.54	3	3.08	3.59
18D	65.1	65.2	2	4.5	3.6	3.7	4	2.8	2.8	2.95	2.62	2.25
18D	65.2	65.3	3.8	4.9	3.5	3.9	3.1	3.31	2.88	3.15	3.18	2.87
18D	65.4	65.5	5.4	6	12.5	8.3	12.7	2.81	2.84	2.42	2.74	2.26
18D	65.7	65.8	3.2	3.6	3.6	2.8	3.1	2.35	2.03	2.17	2.71	3.13
18D	65.8	65.9	2.9	3	2.6	2.7	4.4	1.4	1.64	1.7	1.8	1.58
18D	65.9	66	2.8	2.8	2.8	3.4	4.7	1.86	1.93	1.64	1.63	2
18D	66.5	66.6	1.1	2.5	2.4	3.1	2.5	1.68	1.99	2.42	2.48	1.61
18D	66.6	66.7	3.2	3.4	2.8	3.6	3	2.92	2.83	3.12	3.31	3.53
18D	66.7	66.8	4.1	5.2	4.7	4.4	5	2.95	3.09	3.14	2.75	3.23
18D	66.8	66.9	4.6	5	4.8	4.4	3.7	3.7	3.71	2.53	2.44	2.34
18D	66.9	67	3.6	3.9	3.7	3.8	3.1	1.83	2.09	2.27	2.3	2.4
18D	67	67.1	4	4.4	4.2	4.5	4.2	2.33	2.2	2.23	2.22	2.25
18D	67.1	67.2	3.4	3.4	3.7	4.2	4	2.23	2.23	1.91	1.99	2.55
18D	68.2	68.3	3.2	3.4	3.7	4.5	4.6	1.95	1.85	2.02	2.23	2.56

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	68.3	68.4	4.5	5.4	4.1	5.3	5.2	2.32	2.31	2.12	2.35	2.03
18D	68.4	68.5	6.9	8	7.1	8.8	6.9	2.58	2.67	2.52	2.74	2.73
18D	68.8	68.9	3.6	4.6	4.1	3.7	3.6	2.14	1.94	1.98	2.02	1.93
18D	69.1	69.2	4.6	5.4	4.6	4.8	4.6	2.59	2.93	2.51	2.43	3.01
18D	69.4	69.5	4.7	5	5.2	5.2	5	2.3	2.35	2.19	2.32	2.67
18D	70.2	70.3	3	3.5	2.7	3.3	3.3	1.7	1.79	1.67	1.68	1.48
18D	70.5	70.6	3.4	3.6	5	3.9	5	2.14	2.52	2.66	2.65	3.18
18D	71.1	71.2	1.5	1.6	2.6	2.5	3.3	1.67	1.57	2.19	2.29	1.98
18D	71.4	71.5	3.3	3.6	3.5	4	4.4	1.93	1.83	1.78	1.96	2.41
18D	71.5	71.6	4.1	4.5	4.8	4.9	4.4	1.43	1.41	1.7	1.53	1.68
18D	71.5	71.6	4.1	4.5	4.8	4.9	4.4	1.43	1.41	1.7	1.53	1.68
18D	71.8	71.9	3.8	4.6	6	5.4	5.8	1.97	2.06	2.13	2.11	2.13
18D	72.1	72.2	3.2	5.1	5.5	5.2	7	2.01	2.04	2.35	2.18	2.48
18D	72.2	72.3	4.4	5	5.4	5	5.9	2.49	2.32	2.28	2.35	2.22
18D	72.7	72.8	6.4	6.6	6.6	6.2	6.9	2.38	2.52	2.39	2.34	2.78
18D	72.8	72.9	3.9	4.3	5.1	4.5	4.6	1.77	1.77	1.98	2.02	2.36
18D	73	73.1	5.8	5.9	8.9	8.3	8	1.79	1.82	1.93	1.99	1.96
18D	73.6	73.7	5.2	5.5	5.7	6.2	6	2.42	2.4	2.27	2.42	2.75
18D	73.9	74	4.8	6	5.1	5	5.5	2.13	1.95	2.34	2.26	2.33
18D	74.2	74.3	3.8	4	4.8	4.5	4	2.25	2.27	2.25	2.09	2.23
18D	74.6	74.7	5.5	5.8	6.5	7.5	6.2	1.91	2.07	2	2.01	2.31
18D	74.7	74.8	5.7	9.1	7	7.7	9.9	1.41	1.54	2.18	2.05	2.49
18D	74.8	74.9	4.9	5.7	6.1	5.1	5.7	1.92	2.27	2.24	2.13	2.4
18D	74.9	75	4.6	4.7	4.7	5.5	5	1.51	1.69	2.02	1.81	1.92

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	75	75.1	3.9	4.4	4.9	5.7	5.5	1.48	1.55	1.75	1.85	1.78
18D	75.2	75.3	3.7	3.9	5.8	5.5	5.8	1.72	1.49	2.04	1.91	1.52
18D	75.3	75.4	4.6	6	5.4	6.6	8	1.69	1.98	1.76	2.09	2.6
18D	75.4	75.5	5.6	6.2	5.9	6.2	7.8	1.94	1.88	1.66	1.77	1.79
18D	75.5	75.6	4.3	4.6	6	4.7	4.8	2.11	2.14	1.88	2.06	1.85
18D	76.1	76.2	8.3	8.7	5.7	5.8	6.8	3.46	3.1	1.96	1.78	1.89
18D	77.7	77.8	3.9	4	3.5	4.3	4.2	2.09	2.01	1.96	1.93	1.94
18D	78.1	78.2	4.9	6.3	7.2	6.8	6.7	3.61	3.01	2.67	2.98	3.04
18D	78.2	78.3	5.2	4.8	7.6	5.5	4.9	2.02	2.12	2.48	2.5	2.36
18D	78.4	78.5	6.8	8.4	6.7	6.3	7	2.48	2.4	2.38	2.49	2.57
18D	78.5	78.6	5.3	5.5	12.1	9.3	8.6	2.21	2.23	2.57	2.52	2.28
18D	78.7	78.8	5.9	7.8	13.7	12	13.3	2.96	2.62	2.36	2.38	2.18
18D	78.8	78.9	4.8	5.1	7.8	5.5	4.8	2.25	2.09	2.11	2.24	2.47
18D	79.2	79.3	5.7	6	4.2	4.8	6.5	3.13	3.17	3.39	2.91	2.54
18D	79.3	79.4	3.4	3.5	3.4	3.7	3.1	3.72	3.91	3.98	3.61	3.73
18D	79.6	79.7	3.6	3.9	3.6	3.6	3.9	2.39	2.3	2.98	2.65	2.45
18D	79.7	79.8	3.5	3.7	3.4	4	3.4	2.59	2.57	2.63	2.96	2.76
18D	80.2	80.3	3.4	4.1	5	4.2	5.1	2.39	2.23	2.15	2.44	2.56
18D	80.3	80.4	5	5.4	5.8	4.3	2.3	1.5	1.47	1.47	1.71	2.19
18D	80.5	80.6	3.6	3.8	4.4	3.1	2.5	2.57	2.4	2.26	1.77	2.14
18D	80.6	80.7	2.7	3.9	3.9	2.3	2.6	2.38	2.22	2.19	1.65	1.63
18D	81.3	81.4	4.8	6.1	6	7.7	9.2	2.92	2.87	2.85	2.59	2.89
18D	81.4	81.5	6.4	6.4	7.7	6.8	9.2	2.99	2.87	2.74	2.23	2.43
18D	81.5	81.6	6	6.7	8.2	7.4	8	2.37	1.86	2.24	2.19	2.28

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	82	82.1	2.5	2.6	2.4	2.4	3.1	3.62	2.86	2.87	1.99	2.13
18D	82.6	82.7	4.9	5.8	5.9	3.7	4.7	4.77	4.26	3.76	2.69	3.34
18D	83	83.1	4.4	4.8	4.8	4.5	4.4	2.08	1.98	2.33	1.81	1.74
18D	83.1	83.2	4.8	5.1	4.3	5.4	5.3	3.43	3.16	3.72	2.41	1.83
18D	83.8	83.9	5.7	5.8	5.1	3.6	4.9	3.11	3.25	3.75	2.28	3.08
18D	84.2	84.3	3.2	3.4	3.5	2.8	5.2	3.34	3.27		2.48	2.74
18D	84.3	84.4	3.1	3.6	2.2	3	4.9	3.14	3.4		2.25	2.45
18D	84.4	84.5	6.3	6.7	4.3	5	8.3	2.96	2.85		1.93	2.09
18D	84.5	84.6	5.6	8.4	7.3	6.6	9.2	2.67	2.48		1.78	1.73
18D	84.8	84.9	3.7	4	3.9	3.6	3.4	2.43	2.36	2.78	1.79	1.78
18D	85.3	85.4	2.4	3.6	4.5	5.8	3.8	2.35	1.76		2.69	2.81
18D	85.4	85.5	2.9	3.3	3.8	6.8	4.4	3.35	2.39		2.79	2.86
18D	85.5	85.6	2.8	3.3	4.5	4.3	3.6	3.93	2.63		2.67	2.48
18D	85.6	85.7	3.4	5.5	5.3	5.4	6.2	2.9	2.21	2.29	2.55	2.68
18D	85.7	85.8	3.6	4.9	4.9	5.6	5.3	3.13	1.94	1.9	1.77	1.56
18D	85.8	85.9	2.6	4.4	4.3	5.3	4.9	3.36	2.33	2.32	2.44	2.81
18D	85.9	86	3.3	4.9	4.4	5.5	6.2	3.43	2.1	2.1	2.13	2.28
18D	86	86.1	2.4	4.6	4.4	4.8	4.5	3.4	2.21	2.01	2.09	2.19
18D	86.9	87	2.6	3.5	4.4	1.4	2.1	3.61	4.32	4.07	2.94	3.37
18D	87	87.1	2.3	3	3.3	1.8	3.2	3.15	3.29	3.82	2.65	2.48
18D	88.5	88.6	5.1	5.2		6.5	4.7	1.76	1.82		1.81	2.01
18D	88.7	88.8	6.4	8.2		4.7	4.2	2.92	2.52		2	2.13
18D	89.3	89.4	5.5	6.5	4.8	6	7.4	2.54	2.88	1.8	2.66	2.72
18D	90.3	90.4	3.4	3.9	4.8	4.4	5.9	2.25	2.2	2.24	2.42	2.55

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	90.4	90.5	2.6	3.4	3.6	-1	7.9	2.6	2.26	2.64	3.05	2.97
18D	90.5	90.6	2.6	4.1	4.2	2	7.3	2.62	3.04	3.8	3.87	4.08
18D	90.6	90.7	2.9	3.1	3.8	11.7	12.5	3.1	3.51	3.96	4.31	4.72
18D	90.9	91	3.9	5.7	4.8	4.5	8.4	2.96	3.87	4.2	3.61	5.14
18D	91.4	91.5	4.4	5.5	12.8	1	2.2	3.53	3.6	4.25	1.88	2.08
18D	91.5	91.6	5.1	5.5	11.5	1.9	2.6	2.86	3.47	4.16	2.07	2.14
18D	92	92.1	5.7	6.4	8.3	2.1	2.6	1.99	2.28	2.58	2.03	2.27
18D	92.1	92.2	7.2	7.4	8.2	2.3	3.2	3.33	3.63	3.39	2.08	1.89
18D	92.2	92.3	7.4	7.6	9.2	5	5.7	3.65	3.85	4.01	2.91	2.92
18D	92.3	92.4	5.4	6.3	7.2	2	1.8	4.1	4.38	3.84	2.34	2.43
18D	92.4	92.5	4.2	4.8	6	1.2	1.8	3.34	3.26	3.64	1.32	1.64
18D	92.5	92.6	3.4	3.4	4.4	1.2	1.9	3.01	2.77	3.13	1.28	1.5
18D	92.6	92.7	3.2	3.6	4	1	1.8	2.89	2.88	3.12	1.29	1.7
18D	92.7	92.8	2.5	2.5	3.5	1.1	1.5	2.8	2.79	3.03	1.08	1.21
18D	92.8	92.9	2.2	2.3	3	1.2	1.6	3.98	3.48	3.1	1.17	1.22
18D	93.8	93.9	3.4	3.6	5.1	2.1	2.5	3.7	3.4	3.81	1.4	1.37
18D	94.2	94.3	3.1	3.2	3.3	2.2	2.8	2.99	3.07	3.08	1.13	1.06
18D	94.3	94.4	2.9	4.1	4.3	2.7	3.1	2.94	2.91	3.14	1.8	1.94
18D	94.4	94.5	3.6	4.4	5.8	2.8	3.6	3.31	3.55	3.54	1.66	1.74
18D	94.6	94.7	3.7	3.7	4	2.7	3.1	3.16	2.83	3.92	1.58	1.33
18D	94.8	94.9	2.9	3.1	3.2	3.1	3.8	3.82	4.04	4.48	1.29	1.36
18D	94.9	95	3.3	3.6	4.6	2.6	4.5	3.52	3.93	4.09	1.44	1.57
18D	95	95.1	2.7	4.1	5.2	1.9	2.4	3.68	3.65	4.18	1.66	1.81
18D	95.1	95.2	2.1	3	3.2	1.6	2.5	3.75	3.71	4.04	0.84	0.83

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	95.2	95.3	2.7	3.6	2.7	1.4	2.1	3.95	3.64	3.87	1.25	2.39
18D	95.3	95.4	2.8	3.8	3.7	1.4	2.3	3.31	2.84	3.25	1.39	1.61
18D	95.4	95.5	3.7	5	4.3	1.2	2.2	2.78	2.79	2.93	1	1.1
18D	95.5	95.6	3.6	4.7	4.2	1.1	1.8	3.2	3.04	3.28	1.26	1.21
18D	95.6	95.7	3	3.8	3.3	1.6	2.4	4.12	4.34	3.9	1.78	1.97
18D	95.7	95.8	3.3	3.4	3.7	1.3	2.2	4.06	4.25	4.63	1.38	1.59
18D	96	96.1	3.2	3.8		1.3	2.2	3.4	3.26		1.35	1.6
18D	96.4	96.5	5.5	5.6	5.4	2.3	2.9	2.76	2.76	2.88	1.17	1.47
18D	96.5	96.6	3.7	4	5.1	3.3	4.3	2.53	2.58	3.07	1.29	1.5
18D	96.8	96.9	3	4.1	5.8	1.6	4.1	3.42	3.14	3.39	1.49	1.87
18D	97.3	97.4	5.4	5.6	6.6	3.9	6.1	1.73	2.09	2.59	1.16	1.35
18D	97.5	97.6	6.7	7.4	8.2	5.3	6.1	2.01	2.06	2.4	1.17	1.32
18D	97.6	97.7	5.3	6.9	8.6	5.6	6.1	2.25	2.47	3.08	1.21	1.31
18D	97.8	97.9	4.1	5.2	3.8	7.2	7.8	4.15	4.4	4.62	1.51	1.71
18D	98.7	98.8	3.2	4.1	6.4	2	3.1	3.46	3.9	3.85	1.93	2.21
18D	98.8	98.9	3.1	4.2	6	1.7	3.2	3.68	3.72	3.46	1.5	1.53
18D	98.9	99	3.4	3.8	4.4	1.7	2.7	3.64	3.52	3.68	1.81	1.75
18D	99.9	100	3.6	4.8		3.9	4.2	3.56	3.55		1.46	1.33
18D	100	100.1	3.6	4.6		3.2	3.7	3.58	3.63		1.31	1.48
18D	100.1	100.2	3.3	3.6	2.9	3.3	3.3	3.79	3.22	4.01	1.3	1.72
18D	100.5	100.6	2.7	2.9	4.6	2.5	3.3	3.97	4.01	3.82	1.7	2.08
18D	100.6	100.7	3.2	3.3	4.1	3.3	4.1	3.93	3.74	4	1.71	1.96
18D	101.3	101.4	5.4	6.6	6.6	6.6	8.5	4.48	4.57	4.82	5.37	6.04
18D	101.4	101.5	5.1	5.3	5.5	5.1	5.1	4.23	3.34	3.73	3.68	4.12

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	101.5	101.6	5.1	5.2	5.2	6.9	5.8	3.5	3.37	3.14	3.33	4.09
18D	102.2	102.3	7.1	7.9	7.1	3.5	3.9	3.59	3.08	3.42	1.43	1.57
18D	102.3	102.4	9.2	9.9	10.7	2.8	3.3	4.24	4.05	4.12	1.45	2.11
18D	102.6	102.7	4.6	4.9	5.1	2.7	3	3.35	3.27	3.37	1.36	1.48
18D	102.7	102.8	4.8	5.1	5.9	2.5	2.1	3.45	3.44	3.68	1.18	1.44
18D	103	103.1	7	8.4	9.2	3.2	4.1	4.69	4.81	4.61	1.37	2.69
18D	103.2	103.3	3.6	5.1	5.5	3	2.9	4.43	4.29	4.91	1.18	1.64
18D	103.6	103.7	5	6.3	4.3	3.2	4.2	3.17	3.17	3.6	1.01	1.23
18D	104	104.1	4.9	5.2	5.2	2.5	2.9	2.95	2.87	2.85	1.28	1.81
18D	104.1	104.2	8.7	8.9	8.8	2.2	2.5	4.09	4.02	3.86	1.28	1.29
18D	104.2	104.3	9.6	10.3	10.1	3	3.2	3.51	2.95	4.01	1.41	1.49
18D	104.5	104.6	5.5	6	6.3	3.2	3.1	4.38	4.28	4.65	1.15	1.61
18D	104.6	104.7	6.7	8.2	7.9	2.8	2.8	4.43	3.97	4.34	1.02	1.14
18D	104.8	104.9	6.9	8.4	7.1	2.8	2.9	3.63	3.87	3.7	1.07	1.26
18D	104.9	105	4.3	6	4.7	2.1	3.3	3.24	2.97	3.66	1.34	1.3
18D	105	105.1	5.7	6.4	6.2	2.8	3.2	4.21	4.07	4.28	1.51	1.89
18D	105.4	105.5	5.9	6.1	5.2	2.7	3.1	3.52	3.16	3.73	1.49	1.73
18D	105.6	105.7	5.3	6	6	2.7	2.7	4.03	3.79	4.13	1.07	1.41
18D	105.7	105.8	5.2	5.8	5.6	2.6	3.3	4.45	4.03	3.67	1.19	1.79
18D	105.8	105.9	5.1	5.7	4.2	2.5	2.7	4.14	3.74	3.69	1.09	1.6
18D	105.9	106	6	6.3	4.6	2.5	2.3	3.56	3.22		1.44	2.2
18D	106.1	106.2	7.5	8	7.4	2.3	2.4	4.64	4.88	5.21	1.47	1.71
18D	106.2	106.3	8.2	9.5	7.8	2.4	2.8	4.68	4.5	5.98	1.17	1.46
18D	106.3	106.4	7.1	7.7	5.1	2.8	2.7	4.05	3.04	4.4	1.63	2.48

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	106.4	106.5	6.3	6.8	4.8	2.2	2.7	5.03	4.73	5.05	1.36	2.03
18D	106.6	106.7	8	8.9	7.8	3	3	5.07	5.61	6.37	1.37	1.98
18D	106.8	106.9	8	8.8	8.1	3.1	4.1	4.09	4.06	5.11	1.73	2.13
18D	106.9	107	10.3	12	5.6	3.3	3.5	3.91	4.17	4.66	1.91	2.28
18D	107	107.1	10.2	10.5	4.3	2.9	3.8	3.33	3.44	3.38	1.87	2.48
18D	107.3	107.4	11.2	12.2	11.9	3.3	4.3	3.91	4.35	4.39	1.94	2.35
18D	107.5	107.6	11.4	13.2	13.3	5	6.4	3.63	3.64	3.89	1.86	2.16
18D	107.8	107.9	8.7	9.3	9.6	2.7	3.3	4.05	4.16	4.66	1.92	2.41
18D	108	108.1	5.8	5.8	4.8	2.1	2	4.17	4.09	4.19	1.51	1.62
18D	108.1	108.2	4.9	5	4.6	2.5	2.2	3.32	3.44	3.57	2.04	2.03
18D	108.3	108.4	4.1	5.2	4.6	2.7	2.2	3.49	3.63	3.47	1.49	1.57
18D	108.4	108.5	4.7	6.7	5	3.8	5	3.46	3.52	3.61	2.08	2.15
18D	108.5	108.6	4.4	4.8	3.4	3.5	4	2.51	2.58	3.1	2.91	4.29
18D	108.6	108.7	3.4	3.7	2.5	3.5	3.9	2.02	2.31	2.61	2.02	2.34
18D	108.7	108.8	2.8	4.4	2.1	2.3	2	1.91	2.21	2.26	2.13	2.17
18D	108.8	108.9	2.5	2.8	2.7	3.8	2.8	1.48	1.71	2.27	2.31	2.22
18D	109.1	109.2	3.6	3.8	2.9	3.4	4.2	2.24	1.9	2.17	2.29	2.23
18D	109.2	109.3	3.9	4	2.9	2.8	2.8	2.29	2.2	2.16	2.3	2.67
18D	109.5	109.6	3.3	4.3	2.8	2.8	4.6	2.23	2.42	2.05	1.63	1.63
18D	109.8	109.9	5.7	8.4	2.3	2.8	3.2	1.72	1.65	1.86	1.97	2.15
18D	110	110.1	9.3	11.2	2.8	3.3	3.6	1.59	1.92	1.63	1.72	1.63
18D	110.1	110.2	10.7	12	2.5	3.1	3.8	1.68	1.72	1.59	1.69	1.72
18D	110.2	110.3	6.1	7.1	2.8	3.2	2.7	1.7	1.91	1.54	1.78	2.65
18D	110.3	110.4	2.5	3.9	2	2.9	2	1.46	1.45	1.67	1.97	2.09

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	110.4	110.5	2.3	2.6	1.4	2	1.8	1.11	1.22	2.14	2.11	2.04
18D	110.5	110.6	2.1	3.8	1.3	2.1	2.5	1	1.18	1.95	1.76	2.03
18D	110.6	110.7	2.8	3.3	1.5	2.1	3.3	1.39	1.56	1.5	1.48	1.57
18D	110.8	110.9	3.4	5.2	1.9	2.2	2	1.44	1.51	1.7	1.83	1.76
18D	110.9	111	3.6	3.8	1.8	2.7	3.3	1.57	1.9	1.62	1.5	1.44
18D	111	111.1	3.9	4	1.6	2.4	2.2	1.71	1.56	1.72	1.67	1.65
18D	111.1	111.2	3.2	4.1	1.8	2.5	2.6	1.76	1.88	1.52	1.7	1.62
18D	111.2	111.3	2.7	3	1.9	3.2	2.8	1.72	1.51	1.49	1.6	1.69
18D	111.4	111.5	3.5	3.9	1.5	3	3	1.64	1.54	1.69	1.97	2.13
18D	111.5	111.6	4.4	5.1	1.9	3.2	3.2	1.77	1.43	1.32	1.38	1.49
18D	111.7	111.8	3.6	3.6	1.5	2.7	2.7	1.6	1.73	1.36	1.21	1.3
18D	111.8	111.9	3.7	3.9	1.5	2.7	2.8	1.79	1.93	1.37	1.48	1.45
18D	111.9	112	3.4	3.7	1.9	3.3	3.6	1.67	1.67	1.56	1.72	1.62
18D	112.1	112.2	3.1	3.6	1.7	3.5	3.9	1.58	1.7	1.66	1.92	1.82
18D	112.2	112.3	3.3	3.3	2.6	3.4	4.6	1.76	1.44	1.56	1.85	2.32
18D	112.4	112.5	3.9	4.4	4.9	5.7		1.74	1.89	1.13	1.46	1.78
18D	112.6	112.7	3.8	3.9	4.8	4.8	4.6	1.89	2.07	1.38	1.47	1.63
18D	112.9	113	4.1	5.1	2.9	3.1	3.2	1.85	1.74	1.34	1.36	1.33
18D	113	113.1	5.1	5.9	3.1	3.2	3.6	2.08	1.72	1.3	1.3	1.59
18D	113.1	113.2	4.9	7.5	4.5	3.5	4.8	2.18	2.15	1.4	1.7	2.01
18D	113.2	113.3	4.1	4.9	4.2	3.5	4.5	1.72	2.06	1.1	1.42	1.5
18D	113.5	113.6	2.5	3.8	2.5	4.8	5.4	1.71	1.57	1.41	1.15	1.22
18D	113.6	113.7	2.7	4.8	2.3	3	3.1	1.99	1.81	1.22	1.57	1.64
18D	113.7	113.8	2.8	2.9	2	3.1	3.6	2.48	2.35	1.37	1.55	1.59

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	113.8	113.9	2.7	3.3	1.9	2.6	3.1	2.57	2.93	1.34	1.47	1.7
18D	113.9	114	3.1	3.8	2.1	2.7	2.7	2.45	2.83	1.42	2.34	2.52
18D	114.3	114.4	4.5	5	2.1	2.4	3.1	1.8	1.84	1.42	1.83	1.85
18D	114.6	114.7	3.6	4.8	3.4	3.8	5.5	3.07	3.1	1.7	2.73	2.67
18D	114.7	114.8	3.4	6	3.3	2.7	4.7	2.49	2.49	1.5	1.18	1.68
18D	114.8	114.9	3.1	4.4	2.5	2.6	4.8	1.94	2.1	1.23	1.61	2.18
18D	114.9	115	3.1	4.1	3	2.7	4.3	2.09	2.06	1.26	1.09	1.38
18D	115	115.1	3	3.3	3.6	4.7	5.8	2.3	2.63	1.16	1.4	1.55
18D	115.1	115.2	5.7	6	3.5	4.5	5.5	2.49	2.55	1.32	1.44	1.74
18D	115.2	115.3	6.8	7	2.4	4.3	5.7	2.66	2.7	1.29	1.63	1.86
18D	115.4	115.5	3.9	4.4	3.2	3	3.8	3.12	2.87	1.43	1.17	1.53
18D	115.5	115.6	4.6	4.7	3.4	4.6	4.8	3.6	3.44	1.25	1.48	1.8
18D	115.6	115.7	5.4	6.3	3.2	5.2	5.4	3.48	3.19	1.59	1.56	1.64
18D	115.7	115.8	5.7	6.6	3.1	4.7	5.6	3.05	2.94	1.82	2.09	2.39
18D	115.8	115.9	6.2	6.9	3.3	4.8	5.6	3.47	3.51	1.45	1.17	1.21
18D	116.3	116.4	5.5	5.7	3.8	5.4	4.6	2.97	2.67	1.25	1.66	1.69
18D	116.5	116.6	4.4	5.6	2.2	3.2	3.6	3.53	3.65	1.37	1.15	1.44
18D	116.9	117	4	4.2	1.7	2.5	2.8	4.03	3.01	1.07	1.51	1.71
18D	117.2	117.3	4.5	4.7	2.7	5	4.3	3.4	3.12	1.59	2.78	3.14
18D	117.5	117.6	3.2	3.6	3.8	5.5	6	2.19	2.39	1.27	2.11	2.39
18D	117.6	117.7	3.7	4.2	3.9	4.3	5.5	2.64	2.89	1.15	2.36	2.11
18D	117.8	117.9	3.2	3.3	4.7	5.6	5.9	2.78	2.67	1.05	1.5	1.58
18D	117.9	118	2.9	3.2	4.7	5.6	5.9	2.73	2.59	1.34	2.34	2.49
18D	118	118.1	2.8	3.5	4.5	6.7	6.1	2.48	2.33	1.73	2.68	2.83

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	118.1	118.2	2.9	3.1	3.7	4.6	3.5	2.42	2.42	1.46	1.89	1.87
18D	118.2	118.3	3.2	3.2	2.3	4.3	4.6	2.77	2.97	1.52	2.56	2.58
18D	118.3	118.4	4.1	4.4	1.8	4	4.6	2.97	3.05	1.51	1.85	2.5
18D	118.6	118.7	5.3	5.4	1.8	2.7	2.9	3.66	3.59	1.28	1.43	1.56
18D	119	119.1	4.2	4.7	4.9	5.3	7.2	2.63	2.39	2.38	2.86	3.37
18D	119.1	119.2	5.7	6.3	6.8	4.5	8.9	2.32	2.03	2.94	2.77	3.04
18D	119.2	119.3	5.2	5.7	5.8	4	4.5	2.44	2.71	3.39	3.84	3.53
18D	119.3	119.4	6.2	7.7	5.6	3.4	4.2	3.36	3.74	3.8	3.27	3.63
18D	119.4	119.5	7.6	8.6	4.8	3.4	4.4	3.31	2.96	3.99	3.33	3.18
18D	119.5	119.6	3.7	5.6	4.1	3.8	4.1	3.4	3.66	2.58	2.92	3.28
18D	119.6	119.7	4	5.3	3	3.9	4.5	4.06	3.95	1.56	1.66	2.09
18D	119.7	119.8	4.2	4.9	3.8	4.1	3.6	4.19	3.92	2.38	3.1	1.85
18D	119.8	119.9	4.7	4.8	5.3	4.8	4.7	3.28	3.26	2.95	2.82	3.1
18D	120	120.1	5.3	5.5	8.3	5.1	6.2	2.51	2.42	2.36	2.75	2.66
18D	120.2	120.3	3	3.2	4.2	4.9	4.7	2.22	2.14	2.2	3.08	3.38
18D	120.7	120.8	7.6	8.1	9.4	9.3	9	2.1	2.11	2.6	2.3	2.64
18D	120.9	121	8	8.2	8.6	10.2	10	1.83	1.85	1.74	2.05	2.5
18D	121.3	121.4	5.7	6.6	9.6	4.2	11.6	2.1	2.44	2.38	2.69	3.2
18D	121.5	121.6	2.9	3.6	3.5	8.5	10.6	1.94	2.01	2.19	2.25	2.28
18D	121.6	121.7	2.8	4.2	3.5	3.3	3.2	1.8	1.59	1.93	1.72	1.82
18D	121.7	121.8	3.5	3.9	3.6	3.6	4.8	2.19	2.18	2.17	2.34	2.34
18D	121.9	122	3.9	5	7.5	4.8	10.1	2.16	2.03	2.19	2.37	2.31
18D	122.4	122.5	3.5	3.8	4	4	6.1	1.7	1.55	1.95	2.22	2.11
18D	122.5	122.6	6.2	6.5	7	7.3	8.4	2.65	2.88	3.06	3.09	3.17

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	122.6	122.7	10.6	11.6	10.9	10.3	10.9	2.85	3.03	3.03	3.83	3.62
18D	122.7	122.8	13.3	14.6	14.3	2.4	2.4	3.48	3.38	3.77	1.5	2.3
18D	122.9	123	7.6	8.3	8.9	6.4	11.2	2.45	2.64	2.89	3.2	2.81
18D	123	123.1	10	11.8	12.6	12.7	12.2	2.43	2.45	3.59	2.6	2.95
18D	123.2	123.3	5.1	6.6	6.6	7.1	6.5	1.56	1.5	1.86	1.88	1.84
18D	123.3	123.4	4	4.7	5	5.5	6.2	1.17	1.15	1.27	1.23	1.26
18D	123.4	123.5	2	3.4	4.3	4.4	4.7	1.88	1.61	1.87	1.79	1.55
18D	123.5	123.6	2.3	2.8	4.8	7.2	6.4	2.89	2.41	3.07	2.8	2.6
18D	123.6	123.7	3.3	4.9	5.6	6.3	7.7	1.99	2.05	2.93	2.45	2.59
18D	124.5	124.6	3.2	4	3.9	2.8	3.1	2.36	2.67	2.39	1.85	1.83
18D	124.6	124.7	2.7	3.1	2.6	2.6	4.9	2.08	2.3	2.14	1.87	1.7
18D	124.7	124.8	2.9	3.1	2.7	3.2	5	2.22	2.56	2.35	1.95	2.68
18D	124.8	124.9	2.9	3.2	2.7	4.7	5.4	2.85	2.99	3.13	2.21	1.87
18D	125.3	125.4	8.1	8.5	7.2	11.6	12.4	2.38	2.64	2.61	2.87	2.87
18D	125.4	125.5	15.1	15.5	14.5	17.1	17.2	2.9	2.84	3.17	2.68	2.37
18D	125.6	125.7	3.6	3.9	3.6	5	6.4	2.6	2.53	2.69	2.89	3.17
18D	125.8	125.9	3.5	3.7	4.4	4.7	4.4	3.39	3.31	3.56	3.56	3.44
18D	125.9	126	4.5	4.6	4.7	5.4	5.5	2.33	2.58	2.65	2.54	2.49
18D	126.9	127	5.1	6.2	5.1	4.7	4.8	2.75	2.2	2.42	3.54	2.7
18D	130	130.1	4.3	5.4	1.3	1.8	2.4	2.8	2.59	1.4	1.45	1.43
18D	130.1	130.2	4.3	4.5	1.5	3	3.2	1.42	1.46	1.56	1.65	1.26
18D	130.2	130.3	3.3	3.8	2.1	3.5	4.5	1.69	1.84	2.35	2.69	2.94
18D	130.3	130.4	3.1	3.8	1.9	3.7	4.5	2.12	2.1	1.55	1.98	2.23
18D	131.1	131.2	3.5	3.8	1.4	2.1	2.2	2.73	2.79	1.32	1.34	1.37

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	131.2	131.3	3	3.3	1.4	5.6	6.5	2.72	2.65	1.39	1.7	1.84
18D	131.3	131.4	3.1	4.2	1.4	2.7	2.1	2.57	2.37	1.53	1.86	2.19
18D	131.6	131.7	4.3	5.7	2.1	3.6	4.9	2.69	2.64	1.45	1.65	2.14
18D	131.9	132	3.3	3.7	1.6	2.4	2.4	2.14	2.32	1.41	1.54	1.28
18D	132	132.1	4.3	5.1	1.6	2.5	2.4	2.62	2.61	1.15	1.44	1.54
18D	132.5	132.6	3.4	3.9	1.5	3.6	3.8	1.74	1.55	1.4	1.47	1.6
18D	132.6	132.7	3.7	4.5	1.6	2.7	3.2	1.71	1.72	1.67	1.62	1.85
18D	132.7	132.8	4.1	5.4	1.8	3.4	4.1	1.78	1.86	1.77	1.72	1.96
18D	132.9	133	2.7	2.8		3.1	3.5	1.52	1.51		1.4	1.65
18D	133.1	133.2	3.1	3.5		3.4	3.7	1.98	2.17		1.85	2.19
18D	133.2	133.3	5.2	5.4		2.7	3.6	2.41	2.22		1.16	1.25
18D	133.3	133.4	5.7	6		5.4	6.5	2	1.96		1.37	1.56
18D	133.4	133.5	4.6	6		5	5.4	2.19	2.47		1.49	1.51
18D	133.5	133.6	3.5	4.1		5.7	7.6	2.46	2.72		1.45	2.01
18D	133.6	133.7	3.7	3.9		6.7	7.5	2.23	2.55		1.17	1.31
18D	133.7	133.8	3.5	3.7		6	6.6	2.36	2.39		1.11	1.22
18D	133.8	133.9	4	4.1		5.8	4.4	2.47	2.64		1.92	1.93
18D	134.6	134.7	9.4	10.5		3.5	7.1	3.26	3.06		1.14	1.13
18D	135.3	135.4	4.9	5.1	5.2	5.5	7.2	3.92	3.72	4.55	4.81	4.83
18D	135.4	135.5	7	8.2	7.1	9.1	10	3.38	3.67	4.42	3.61	3.91
18D	135.5	135.6	3.7	4.8	4.9	5.7	5.7	2.24	2.33	2.99	2.33	2.26
18D	135.6	135.7	2.3	3.4	3.3	3.2	8.3	2.11	2.32	2.54	2.95	3.56
18D	136	136.1	4.2	4.3	3.9	4.9	4.7	2.05	2.11	2.41	2.18	2.57
18D	136.1	136.2	2.3	2.6	2.3	2.8	4.4	1.25	1.53	1.66	1.62	1.88

Road Section ID	Start	End	Rutting					Roughness				
			Pre-flood	Post-flood	2012	2013	2014	Pre-flood	Post-flood	2012	2013	2014
18D	136.2	136.3	2.3	2.5	2.9	2.9	9.1	1.61	1.71	1.79	2.01	1.98
18D	136.6	136.7	4	4.1	4.6	5.3	7.8	2.27	2.42	2.84	2.81	3.08
18D	137.8	137.9	1.6	2.7	4.4	2.5	2.5	1.12	1.67	2.53	2.11	2.42
18D	137.9	138	3.9	3.8	4.9	2.3	3.7	1.43	1.53	1.69	1.67	1.89
18D	138.2	138.3	1.7	2.1	3.3	2.5	3.6	2.6	2.25	2.24	2.12	1.74
18D	138.6	138.7	2.9	3.4	1	1.4	1.5	1.58	1.7	1.46	1.22	1.28
18D	138.7	138.8	2.4	2.9	1.2	1.6	1.9	1.44	1.59	1.44	1.48	1.45
18D	138.8	138.9	3	3.6	2.2	2.5	3.4	2.18	1.94	1.87	1.87	1.9
18D	139.2	139.3	2.9	3.2	2.2	3.2	4.6	2.64	2.39	2.02	1.85	2.04
18D	139.3	139.4	2.4	2.6	2.3	5.3	3.6	1.83	1.76	1.64	1.7	1.4
18D	139.4	139.5	3.9	4.2	3	5.5	3.8	1.57	1.23	1.05	1.05	0.92
18D	139.8	139.9	1.5	1.8	1.4	2.9	2.4	1.41	1.5	1.79	1.83	2.16
18D	139.9	140	1.5	1.8	0.8	1.8	2.6	1.57	1.75	1.25	1.48	1.45
18D	140.1	140.2	2.1	2.2	0.8	1.4	2.3	2.77	2.11	1.67	1.22	1.49
18D	140.3	140.4	1.8	1.9	0.7	1.8	2.8	1.74	1.9	1.85	1.38	1.58
18D	140.4	140.5	1.6	2.2	0.6	1.4	2.4	1.92	2.28	1.54	1.67	2.11
18D	140.5	140.6	1.6	2.3	0.6	0.9	2	2.11	2.08	1.8	1.49	2.22
18D	140.6	140.7	2.3	2.4	0.8	1.9	3.6	2.55	2.17	2.08	1.84	2.58
18D	140.7	140.8	1.6	1.9	1	1.7	2.7	3.18	2.94	2.71	2.65	2.8
18D	140.8	140.9	1.8	2.1	1.9	3.7	3.6	2.49	2.37	3.17	3.44	3.78
18D	141	141.1	2.7	3.1	2.1	2.1	2.2	2.34	2.41	2.47	2.39	2.37