University of Southern Queensland
Faculty of Engineering and Surveying

Forensic Investigation of Pavement Failures

A dissertation submitted by

Richard Boyd Smith

in fulfilment of the requirements of

Courses ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

Submitted: October, 2004

Abstract

Forensic engineering can be defined as the application of the engineering sciences to the investigation of failures or other performance problems, with a focus on uncovering causes so that improved facilities can be engineered.

The investigation of road pavement failures can be done in a systematic manner using the principles of forensic engineering. The methodology developed in this project has been based on similar work previously conducted in various locations, mainly in Australia and the United States. The focus of this project is on creating a systematic, and yet simple and easy to understand guide that is flexible enough for use in a variety of situations.

The investigation methodology developed was trialled for several pavement failures in Southern District of the Queensland Department of Main Roads, to evaluate the effectiveness of the method for everyday practical use.

It was found that the method was good as a general guide, particularly for people with limited experience in the area of pavements and road engineering. The expertise of the investigator is however an important factor in correctly diagnosing the pavement failure cause and determining the appropriate rehabilitation treatment, and so the use of experienced personnel would generally be expected to lead to a better diagnosis.

The study showed that for most pavement failures it is necessary to carry out some form of materials sampling and testing, if conclusive evidence regarding the failure cause is to be found. Otherwise, interpretation of the limited data available can be difficult, or even misleading, resulting in improper conclusion being made regarding the failure cause.

The pavement failure investigation methodology developed in this project can serve as a useful guide for the investigation of pavement failures. The method, combined with the experience of the investigator and adequate materials investigation, will help to ensure that the cause of a pavement failure can be reliably determined.

University of Southern Queensland

Faculty of Engineering and Surveying

ENG4111 & ENG4112 *Research Project*

Limitations of Use

Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Richard Boyd Smith
Student Number: 0011122174

Signature

Date

Acknowledgments

Firstly, I would like to acknowledge my family for their support over all the years, without which this dissertation would not have been possible.

Secondly, I would like to thank any personnel at the Queensland Department of Main Roads and RoadTek who helped me with my project, either directly or indirectly. Special thanks go to the following people:

Dave Seefeld, for his general guidance, and knowledge about pavements and how they fail. Chris Brincat, for his information about materials testing and feedback on the investigation method.

John Devlin and Selwyn Leslie, for their help in accessing road asset management data from the ARMIS system, using Chartview. Eugene Maliszewski and Bob Thompson, for providing copies of deflection testing results for use in this project.

Last, but not least, I would sincerely like to thank Associate Professor Ron Ayers, my project supervisor. Without his wisdom, advice and critical feedback the project would have been much poorer.

Richard Smith

University of Southern Queensland
October 2004

Contents

Chapter 2 6

Background and Literature Review

List of Figures

Chapter 2

Background and Literature Review

Section 2.1 Queensland Department of Main Roads

Section 2.3 Types and Causes of Road Pavement Failure

2.9 Fatigue/structural cracking: a) Due to inadequate edge support (WSDOT), b) On a poorly supported road widening, accompanied by rutting (WAPA)

2.10 Severe structural cracking, allowing easy moisture entry into the pavement (WSDOT)

2.11 Joint cracking due to poor joint location and construction practices (WSDOT)

2.12 Longitudinal cracking: a) Possibly caused by ground movement, b) In the wheelpaths, as an onset to structural cracking (WSDOT)

2.13 Slippage cracking near to a bus stop, caused by acceleration and braking (WSDOT)

2.14 A depression in the shoulder of a road, probably due to local moisture entry resulting in weakness of the pavement and subgrade (WSDOT)

2.15 A patch placed over an area of localised pavement distress (WSDOT)

2.16 Polishing of surface aggregate, with a lack of surface texture, but no excess binder (HAPI)

2.17 Potholes: a) As a result of moisture entry due to fatigue cracking of the pavement surface, b) Occurring only in the surface course, with the pavement base still intact (WAPA)

2.18 Rutting: a) On a road widening, caused by inadequate compaction of the affected area (WAPA), b) On recently completed asphalt, probably due to instability and poor mix design (WSDOT)

2.19 Shoving: a) Major localised deformation, b) Near an intersection (HAPI)

2.20 Ravelling: a) Of a 20 year old surfacing, due to ageing and embrittlement of the binder (HAPI), b) Within the wheelpaths of a road (WSDOT)

2.21 Stripping: a) Of a recently completed seal, b) In both wheelpaths on an old seal

Section 3.7 Destructive Materials Sampling and Testing

Section 3.9 Determine Best Rehabilitation Treatment

Chapter 4
Investigation of Pavement Failures

Section 4.1 Bowenville-Dalby

Section 4.2 Brookstead - Pampas

Section 4.3 Drayton Connection Road

Section 4.7 Yaralla Deviation

Appendix E

Bowenville-Dalby: Test Results

Appendix F

Gatton Bypass: PAVDEF Deflection Testing Results

Appendix G

Gatton Bypass: FWD Deflection Testing Results

List of Tables

Chapter 2
Background and Literature Review

Section 2.1 Queensland Department of Main Roads

Section 2.5 Pavement Failure Investigation Methodologies

Chapter 3
The Investigation Method

Section 3.3 Plan the Investigation

Chapter 4 – Investigation of Pavement Failures

Section 4.1 Bowenville-Dalby

Section 4.3 Drayton Connection Road

Section 4.4 Gatton Bypass

Section 4.5 Tabletop Turnoff

Section 4.6 Warrego Highway through Toowoomba

Section 4.7 Yaralla Deviation

Appendix C Project Methodology and Timelines

Appendix D Safety Issues and Resource Planning

Appendix F Gatton Bypass: PAVDEF Deflection Testing Results

Appendix H Warrego Highway through Toowoomba: PAVDEF Deflection Testing Results

Appendix I Yaralla Deviation: PAVDEF Deflection Testing Results

Glossary

AADT	Average Annual Daily Traffic, the typical number of vehicles using a section of road each day
ALD	Average Least Dimension
ARRB TR	Australian Road Research Board Transport Research
ARMIS	A Road Management Information System, used by the Queensland Department of Main Roads for the collection and storage of linear and spatial road-related data
CBR	California Bearing Ratio
C_F	Curvature Function, $C_F = D_0 - D_{200}$
Ch.	Chainage, length along a road measured from a specific start point
CL	Centreline
CV	Coefficient of variation, CV = SD / X x 100
DCP	Dynamic Cone Penetrometer
DGA	Dense Graded Asphalt
DOS	Degree of Saturation
D_0	Rebound Deflection, D_0 = Maximum Deflection – Residual Deflection
D_X	Deflection measured X mm away from the loading point
D_r	Representative Deflection, which for a homogenous section corresponds to the 90% highest Rebound Deflection (D_0)
D_R	Deflection Ratio, $D_R = D_{250} / D_0$ (may be a percentage)

EB Eastbound

ESA Equivalent Standard Axles

FWD Falling Weight Deflectometer

GNP Gross National Product

GPR Ground Penetrating Radar

HIPAR Hot In-Place Asphalt Recycling

HSS Highly Stressed Seal

HV Heavy Vehicles

IL Inner Lane

IWP Inner Wheelpath

LL Liquid Limit

LS Linear Shrinkage

MDD Maximum Dry Density

MRS Designation of Queensland Main Roads Standard Specifications

NAASRA National Australian Association of State Road Authorities, now known as Austroads

OECD Organisation for Economic Cooperation and Development

OGA Open Graded Asphalt

OMC Optimum Moisture Content

OL Outer lane

OWP Outer Wheelpath

PAVDEF A variation of the deflectograph deflection testing machine, used in Queensland

PI	Plasticity Index, PI = LL – PL
PL	Plastic Limit
PMB	Polymer Modified Binder
PRP	Permanent Reference Point
PSD	Particle Size Distribution (Grading)
RLT	Repeat Load Triaxial
RS&E	Road System and Engineering, a specialist division of the Queensland Department of Main Roads
RTFO	Rolling Thin Film Oven
SAM	Strain Alleviating Membrane
SAMI	Strain Alleviating Membrane Interlayer
SCRIM	Sideways Coefficients Routine Investigation Machine
SD	Standard deviation of deflection parameter for a homogenous section
SFC	Sideways Force Coefficients
SMA	Stone Mastic Asphalt
TxDOT	Texas Department of Transportation
WB	Westbound
X	Mean of deflection parameter for a homogenous section

Chapter 1

Introduction

1.1 The Importance of Road Pavements

Roads are vitally important for the Australian economy and people. As outlined in the *Austroads Pavement Strategy 2001-2004* (Austroads, 2002a), Australia has the largest proportion of road freight of any OECD country, measured per capita or per unit of GNP. This road freight, measured in tonne-km, has tripled since 1979, and is predicted to double between 2000 and 2015.

The quality and performance of road pavements significantly affects the following:

- The safety and comfort of road users
- The effective management of the road freight task
- The load-carrying capacity and hence the life of the road
- The type and extent of wear and tear on vehicles, especially heavy vehicles
- The impact on amenity of the surrounding natural and built environment

According to *Austroads Pavement Strategy 2001-2004* (Austroads, 2002a), the total value of road pavements in Australia and New Zealand is about A$50 Billion, or one third of the total value of road infrastructure. Pavement works cost about A$3 Billion per year, or nearly half of the total annual road expenditure. Therefore, road pavement deterioration imposes a high financial burden on road agencies, particularly if deterioration occurs prematurely.

To ensure the risk of premature deterioration is minimised, it is necessary to use the best practice pavement technology in the planning, design, construction, maintenance, rehabilitation, and operation of the road system.

A greater knowledge of what constitutes best practice pavement technology can be achieved by examining pavements that have failed prematurely, with the focus being on determining the cause(s) of the failure so that it can be prevented in the future, and designing a rehabilitation treatment that minimises any further deterioration.

1.2 Forensic Engineering

A forensic engineer was originally considered as a professional engineer who dealt with the engineering aspects of legal problems (Carper, 1989). However, in recent years the meaning of the term has changed so that a more appropriate definition is as given by Victorine et al. (1997), who stated that forensic engineering was:

> "The application of the engineering sciences to the investigation of failures or other performance problems… Forensic engineering attempts to…uncover the causes of failures so that improved facilities can be engineered."

The systematic investigation principles of forensic engineering can and have been applied to the examination of road pavement failures. This is desirable because a systematic investigation methodology will reduce the reliance on the experience of the investigator in determining the cause of a pavement failure, and help ensure that the cause is reliably determined.

1.3 Aim and Objectives of the Project

This project, Forensic Investigation of Pavement Failures, seeks to develop and apply a forensic method for the investigation of pavement failures in the Southern (Toowoomba) District of the Queensland Department of Main Roads, with scope for use of the method in other areas as required.

In order to achieve this aim, specific objectives were developed and will be discussed further. These objectives are also detailed in the Project Specification and Project Methodology, in the appendices.

Firstly, it was necessary to research background information on forensic engineering and pavement engineering, including the history and methods of forensic engineering, the application of forensic engineering in the road engineering area, and pavement assessment, testing and failure modes. This was a crucial part of the project, since knowledge of the subject area was essential to ensure that the best information was used when carrying out the project, and to minimise duplication of work previously carried out.

The next step was to develop a systematic method for the forensic investigation of pavement failures, which was applicable to the Southern (Toowoomba) District of Main Roads. This was done by utilising the information found in the background and literature review, while incorporating variations to ensure the usefulness and flexibility of the method.

In conjunction with Main Roads staff, a number of failing pavements within the Toowoomba District were selected. This was completed by finding currently failing pavement sections that would be suitable for investigation.

Once the investigation method was developed and the test sites were selected, the method was applied to the sites, taking account of variations required due to the amount of information available and testing carried out.

The information gathered from the application of the method to these sites was examined to decide if adjustment to the original method was needed. It was also examined whether the forensic method developed was appropriate for use in the Toowoomba District, from both a technical viewpoint and an economic viewpoint, and its applicability to the future.

Finally, and perhaps most importantly, the findings of the project were required to be presented to other professional peers, in both oral form (at the September Project Conference), and in written form (this dissertation).

1.4 Dissertation Overview

This dissertation was split into five chapters, with the listing being generally in the chronological order in which the work and information was developed.

Chapter 1 is the introduction, and also lists the aim and specific objectives of the project, and provides an overview of the dissertation.

Chapter 2 is the background and literature review, and serves as an introduction to the topic for people not familiar in this area. This chapter contains details about the Queensland Department of Main Roads, engineering failures and the application of forensic engineering to this area, the main types and causes of road pavement failure, types of pavement testing, pavement failure investigation methodologies previously developed, and some sample case studies.

Chapter 3 is the chapter in which the investigation method is developed. An outline of the overall process is given, followed by specific details for each of the eight investigation steps. These investigation steps are as follows:

- Plan the Investigation
- Review Documents and Literature
- Interview Personnel
- Non-destructive Condition Survey
- Destructive Materials Sampling and Testing
- Determine Probable Cause(s) of Failure
- Determine Best Rehabilitation Treatment
- Report on Outcomes

In addition, a simple methodology for the investigation of pavement failures at the network level was developed.

Chapter 4 is the chapter in which the investigation method previously developed is applied to several sample pavement failures in the Southern District of the Queensland Department of Main Roads, illustrating how the method could be used in real life, and as a test of the effectiveness of the method that was developed.

Chapter 5 is the chapter in which the conclusions regarding the project are listed, including a discussion on how well the project objectives were achieved, and further work that could possible be carried out in the future.

Chapter 2

Background and Literature Review

2.1 Queensland Department of Main Roads

2.1.1 Introduction

The Queensland Department of Main Roads is the custodian of the state-controlled road network, which has a total length of about 34,000 km. These roads consist of the main highways and other major connecting roads throughout the state.

In addition, the Department manages the national highway network in Queensland on behalf of the Federal Government, which is responsible for the funding of these roads. These highways are considered crucial for transport on a national basis.

The work of the Department includes the planning, design, construction and maintenance of these roads and related infrastructure such as culverts and bridges. In addition, the Department has primary responsibility for the management of activities that occur in the road reserve, and interacts with local councils.

The network managed by Main Roads, while only consisting of about 20% of the state's road network by length, carries 80% of the traffic. For this reason, it is very important to the state's economy and regional development, and increases the quality of life for all Queenslanders by giving access to jobs, health, education and other services.

Further information about the Department is available from the website: http://www.mainroads.qld.gov.au/

2.1.2 Organisation

The Department is decentralised, with offices throughout the state. The state is split into four regions, which are each made up of smaller districts. In addition to these district offices, other important parts of the organisational structure include RoadTek (a commercialised business unit) and Road Systems and Engineering (RS&E), a specialist division based in Brisbane.

Table 2.1 – Region and districts of the Queensland Department of Main Roads

Region Name	District Name	Head Office
South East Queensland	South Coast Hinterland North Coast Hinterland Metropolitan	Nerang Gympie Spring Hill
Southern Queensland	Wide Bay Southern South Western Border	Bundaberg Toowoomba Roma Warwick
Central Queensland	Central Central Western Mackay Central Highlands	Rockhampton Barcaldine Mackay Emerald
North Queensland	Northern North Western Peninsula	Townsville Cloncurry Cairns

2.1.3 Southern (Toowoomba) District

This project will be conducted in Southern District, which has its office in Toowoomba. The main challenge of this district is to manage the ageing road network to meeting growing needs of agricultural, tourist and industrial traffic, while interacting with the many local governments contained within its boundaries.

Southern district has about 3,100 km of state-controlled roads within its boundaries, including low traffic district roads, moderately trafficked regional and state-strategic roads, and about 385 km of National Highways, which are funded by the Commonwealth Government. The most important roads within the district are shown in the table and figure.

Figure 2.1 – Southern District, Queensland Department of Main Roads

Table 2.2 – Important state-controlled main roads within Southern District

Road Name	Description	Classification	Length (km)
Warrego Highway	Ipswich-Toowoomba Toowoomba-Dalby Dalby-Miles	National National National	66 84 106
New England Highway	Yarraman-Toowoomba Toowoomba-Warwick	State Strategic State Strategic	118 34
Gore Highway	Toowoomba-Millmerran Millmerran-Goondiwindi	National National	79 49
Moonie Highway	Dalby-St George	Regional	50
D'Aguilar Highway	Kilcoy-Yarraman Yarraman-Kingaroy	State / Regional State / Regional	58 45
Burnett Highway	Nanango-Goomeri	State Strategic	43
Brisbane Valley Highway	Ipswich-Harlin	State Strategic	84
Bunya Highway	Dalby-Kingaroy Kingaroy-Goomeri	Regional Regional	109 52

2.1.4 The Impact of Pavement Failures

Reeves (2001) provides information about the magnitude of pavement failure on roads managed by the Queensland Department of Main Roads, including the following facts:

- The cost of pavement failures to the Queensland Department of Main Roads is about $10 million per year, at a conservative estimate.

- Of about $3 billion in funding spent on pavements in last 10 years, about $300 million (10%) has not given optimum value since remedial treatment was necessary to allow pavement to reach its intended design life.

- About $10 million is spent on unplanned costs each year due to design or investigation inadequacies, which result in costly construction changes and claims.

As indicated by this information, the greater understanding of pavement failures that could be gained from detailed investigations could be valuable in reducing the costs associated with pavement failures in the future.

2.1.5 Pavement Failure Investigations

The investigation of pavement failures in Southern District of the Queensland Department of Main Roads is often done in a relatively informal manner. While there are specialist services for the investigation of pavement failures available from Road Systems and Engineering (RS&E) divisions, these are often not utilised.

The main reason for this choice of action is that the cost associated with the investigation is not justified by the importance of the road on which the failure is occurring, or the magnitude of the failure. There may also be a reluctance to spend money on a pavement failure investigation, instead of using these funds for rehabilitation work.

Because of these factors, there is a reliance on the past experience of the investigator in evaluating the failure, determining the testing required to be done, and making a final decision regarding the appropriate rehabilitation treatment. This means that the failure may be incorrectly diagnosed if the wrong testing is selected, or the investigator's experience is lacking.

The development of a systematic method for investigating pavement failures will help to ensure that even if the investigation is carried out by inexperienced staff, there is a reasonable chance of success in diagnosing the problem, and determining the best rehabilitation treatment.

2.2 Engineering Failures and Forensic Engineering

2.2.1 Introduction

There are many examples of engineering failures, and while most failures may not be catastrophic, those of a catastrophic nature are more widely known. Examples of recent catastrophic failures that have made media headlines include the following:

- Columbia Space Shuttle, USA, 2003
- World Trade Centre Buildings, New York, USA, 2001
- Air France Concorde Crash, France, 2000
- Longford Gas Plant Explosion, Victoria, Australia, 1997
- Oklahoma City, Oklahoma, USA, 1995

While none of the above examples is in the area of road engineering, they illustrate that failures occur in all areas of engineering.

2.2.2 Engineering Failure

The simplest definition of an engineering failure is similar to that provided by Victorine et al. (1997) and Campbell-Allen (1987), which is that for an engineering structure, there is:

> "An unacceptable difference between observed and expected performance"

This definition is very simple and indicates that a failure need not be catastrophic to be considered a failure. This definition is especially useful for the area of road engineering, since failures in this area are seldom catastrophic, although they are still important.

2.2.3 Forensic Engineering

Initially Forensic Engineering was a term used to describe the engineering aspects of legal problems. For example, the Cambridge English Dictionary defines 'forensic' as:

> "Belonging to courts of justice, or to public debate; used in courts or legal proceedings".

This meant that the term was used only to describe investigations where legal proceedings were a major part, such as the collapse of a building or bridge where lives were lost. Considering this, Carper (1989) stated that:

> "[A] Forensic engineer is a professional engineer who deals with engineering aspects of legal problems".

Campbell-Allen (1987) considered that forensic engineering is:

> "[The] application of the art and science of engineering in the jurisprudence system, requiring the services of legally qualified professional engineers."

He acknowledged that the definition was too narrow for the Australian scene, since most engineers do not have legal qualifications

However, in recent years, the term "forensic engineering" has evolved to a wider meaning related to any form of systematic engineering failure investigation.

Two recent American (ed. Rens 1997, 2000) conferences and a British conference (ed. Neale, 1998) on Forensic engineering have dealt with a broad range of failures, and not only those involving loss of life or legal proceedings. These conferences have highlighted the fact that forensic engineering is now a term that can be applied to any systematic engineering failure investigation.

In light of this, a more appropriate definition of forensic engineering is that given by Victorine et al. (1997), who stated that forensic engineering is:

> "The application of the engineering sciences to the investigation of failures or other performance problems…Forensic engineering attem pts to…uncover the causes of failures so that improved facilities can be engineered."

The term "forensic engineering" has not been as widely used in the United Kingdom and Europe in the past, but is becoming more commonly used, with several universities across Europe now offering courses in Forensic Engineering.

It should be noted that while most pavement failures occur over a period of days or months and could not be classed as catastrophic, the incidence of litigation and legal issues relating to pavement failures is increasing. This is probably due to claims resulting from damage to vehicles or personal injury.

2.2.4 Tasks carried out by a Forensic Engineer

The tasks carried out by a forensic engineer and the qualities required of such a person vary depending on the area in which the investigation is being carried out. They are also dependent on whether the primary focus of the investigation is the investigation of the failure, or dealing with the engineering aspects of a legal case.

Victorine et al. (1997) stated that a forensic engineer investigates failures related to construction or design, and determines the causes, and sometimes responsibility. It was considered that they should be an acknowledged expert in the field, with a detailed knowledge of structure being investigated, including aspects relating to design, construction, operation, codes and test methods.

It was also stated that tasks performed by a forensic engineer might include the review of all documents relating to the project. It is important that the engineer develops an intuitive understanding of why structures fail, while not relying on past experience too early in the investigation.

Both Carper (1989) and Campbell-Allen (1987) added that in addition to investigating the cause of failures, a forensic engineer might be required to prepare reports and provide testimony or expert opinions regarding the case.

2.2.5 The Importance of Forensic Engineering

Forensic Engineering is of increasing importance for three major reasons, and these points will be discussed in the following paragraphs.

Firstly, many engineering structures constructed in the past are experiencing deterioration and need repair or replacement, and the accurate determination of failure mode will usually result in the most cost effective method of repair or rehabilitation. This has been succinctly stated by Victorine et al. (1997) as follows:

> ‘The practice of forensic engineering within the field of infrastructure management has grown in response to the increasing number of facilities needing repair or replacement..Forensic engineering has become increasingly important in recent years, given that the repair and replacement of a deteriorating infrastructure often depends on the skills and knowledge of forensic experts.”

Secondly, the advancement of engineering knowledge will always be assisted by an understanding of failures. This advancement can lead to improvements in design, construction and maintenance. This point was stressed by Campbell-Allen (1987).

This was especially true before the development of theoretical models for describing the behaviour of engineering structures, when trial and error combined with past experience was required to extend the boundaries of knowledge.

While structural analysis and increasingly elaborate models of engineering structures have somewhat reduced this need, the investigation of failures still is of great assistance in advancement of engineering knowledge.

As mentioned by Campbell-Allen (1987), this is true in recent years, with some design rules being developed by learning from failures (e.g. box-girder bridges), which could be considered as unplanned full-scale experiments.

Thirdly, society is becoming more litigious. Campbell-Allen (1987) has stated that:

"..as a consequence of increase in number and extent of claims and law cases related to engineering failures, forensic engineering is becoming more common."

As can be seen from the above points, forensic engineering is quite important from both engineering and legal perspectives. This means that the practice of forensic engineering will grow in importance in the 21st century.

2.2.6 Learning from Failures using Forensic Engineering

Campbell-Allen (1987) covers this general topic in some detail. He observes that much can be learned from the majority of failures that are non-catastrophic, in addition to the less common catastrophic failures.

If learning is to be achieved, the information should be available to interested parties, but this can be quite difficult to achieve in practice, due to difficulty disseminating information, because of limitations in communications technology and other factors. Since the article was written, advances in communications technology have been great. However, information may still not be made available for the following reasons:

- Legal proceedings may be quite long, and information may not be available for release until its conclusion.

- There may be a reluctance to change established procedures and so information may be selectively released to aid this.

- People may be reluctant to criticise or blame others, because of the chance of being charged with libel.

Campbell-Allen (1987) has classed failures as either technical or procedural failures, and the application of lessons learned from failures as either general or specific.

General application may be in the form of changes to codes of practice, industry norms or legislation. As mentioned, all of these have both advantages and disadvantages, and should be carefully considered before carrying out changes.

Specific application can be by correction of technical defects, due to reports on similar structures. However, this may leave the structure vulnerable to other deficiencies.

Campbell-Allen (1987) concludes that:

> 'There is a need to ensure records of accidents/failures are published so they are accessible to all engineers, and there must be a willingness to do this for it to be successful".

He states that information of a forensic nature should be available, in the form of case studies, reports on errors in methods or procedures, and the types of a certain distress. He acknowledges that the motivation for this to occur must come from within the industry and profession if it is to be successful.

He provides some information in his article about information systems already set up in other countries that could perhaps be adapted to Australian conditions.

2.2.7 Steps in a Forensic Engineering Investigation

This section covers the general steps in a forensic engineering investigation, and further detail about how this has been applied to the investigation of pavement failures will be discussed in another section.

Several key points regarding a forensic engineering investigation, as stated by Victorine et al. (1997) are as follows:

- The investigation is the process by which information is gathered to determine the probable cause of failure.

- There is seldom a single cause of failure and often there is a complex interaction of causes. Conflicting opinions as to the true cause often exist, which leads to only the most probable cause being reported.

- The investigation must determine if failure is due to design, construction, materials, a combination of these, or other factors.

- The investigation may be required to assign responsibility for failure, and any other contributing factors.

The general steps in a forensic investigation are defined by Victorine et al. (1997) and Carper (1989). In addition, a report detailing the development of a formal forensic investigation procedure for pavements in Texas, USA, has been prepared by Crampton et al. (2001).

In Australia, the Queensland Department of Main Roads details a general methodology for investigating the rehabilitation of pavements in their Pavement *Rehabilitation Manual* (Queensland Transport, 1992).

All of the above are quite similar in the general principles developed. It can be seen that the basic steps in any forensic engineering investigation consists of the following steps:

- Plan the Investigation
- Retrieve all possible data required for solution of the problem
- Analyse the data obtained
- Conclude as to the probable cause(s) and responsibility for the failure

These steps are quite general and can be applied to any investigation ranging from serviceability problems to severe failures. These steps as above should provide a good basis for the investigation method developed within this project, with modifications as required.

2.3 Types and Causes of Road Pavement Failure

2.3.1 Introduction

This project is being developed for use in Southern District of the Queensland Department of Main Roads. In this district, and in Australia, the majority of state-controlled roads consist of a spray seal or asphalt surfacing overlying an unbound granular pavement. This may be considered a flexible pavement.

Since the majority of pavements in the district are of this type, the types and causes of pavement failure will be confined to these types of road pavement, with no discussion of failures in rigid (e.g. concrete) pavements.

According to Woods and Adcox (2002), pavement failure may be considered to be either a structural, functional, or materials failure, or a combination of these types.

Structural failure is the loss of load carrying capability, where the pavement is no longer able to absorb and transmit the wheel loading through the fabric of the road without causing further deterioration.

Functional failure is a broader term, which may indicate the loss of any function of the pavement such as skid resistance, structural capacity, and serviceability or passenger comfort. Materials failure occurs due to the disintegration or loss of material characteristics of any of the component materials.

Austroads (2000a) categorises the main types of pavement failure as either deformation failures or surface texture failures.

Deformation failures include corrugations, depressions, potholes, rutting and shoving. These failures may be due to either traffic (load associated) or environmental (non load associated) influences. It may also reflect serious underlying structural or material problems that may lead to cracking.

Surface texture failures include bleeding and flushing, cracking, polishing, stripping and ravelling. These failures indicate that while the road pavement may still be structurally sound, the surface no longer performs the function it was designed to do, which is normally to provide skid resistance, a smooth running surface and water tightness.

Other miscellaneous types of pavement failure that cannot be so easily categorised include edge defects, patching and roughness.

2.3.2 Information Sources

The information in the following sections on specific types of failure has been summarised from the many sources that provide generally similar information. The most important references are as follows:

Australia

- *Guide to Selection of Road Surfacings* (Austroads, 2000a)
- *Pavement Management* (Dowling, 1998)
- *Risk Profiles in Pavement Rehabilitation* (Ramanujam, 2001)
- *Pavement Rehabilitation Manual* (Queensland Transport, 1992)

United States

- *Appendix A -Pavement Failure Identification, Identifying and correcting pavement failures* (APAI, 2003)
- *Flexible Pavement Rehabilitation Manual* (CalTrans, 2001)
- *Appendix A - Evaluating Premature Distress of Hot Mix Asphalt and Portland Cement Concrete Pavements* (Demos, 2003)
- *Pavement Distress* (WSDOT, 2004)

2.3.3 Moisture and Road Pavements

In many pavement failures, excess moisture is the main cause of failure or a contributing cause, so moisture and its relation to road pavements will be examined.

Effect on Strength and Stiffness

Queensland Transport (1992) and Queensland Department of Main Roads (1993) both give information on the effect of moisture content changes on the strength and stiffness of pavement materials.

The basic conclusion reached is that excess moisture reduces the strength and stiffness of pavement materials, being worse for the subgrade material, than for the subbase or base. Excess moisture and particularly high degrees of saturation result in significant pore pressures within the material.

Depending on the degree of saturation, failure may occur as any of rapid shear/bearing failure, premature rutting, lifting of wearing course due to positive pore pressures, or embedment of cover aggregate due to weak base.

It can be seen that for nearly all types of pavement failure, moisture is often the primary or a contributing cause of failure.

Possible Sources of Moisture Entry

Queensland Transport (1992) and Queensland Department of Main Roads (1993) both give information regarding possible sources of moisture entry into a pavement and other factors that influence the moisture in a road, and these will be discussed.

Moisture entry through the surface may be caused by unsealed shoulders, inadequate pavement surface drainage during construction, exposure of surface to rain during construction, or porous or open graded asphalt.

Moisture entry from the side may be caused by pondage in pits or poorly constructed surface drainage, and lateral movement of water into pavement.

Moisture entry due to construction practices could be a result of using a boxed or trenched pavement, the pavements being left primed for extended periods, use of high moisture content for compaction, or excessive watering of surface for surface finishing requirements.

Other factors affecting the moisture in a pavement include the general drainage condition, such as the effectiveness of drainage structures, shoulder crossfall and condition, longitudinal grade, and whether the pavement is constructed on cut or fill.

The characteristics of the water table, such as whether it is static or variable, and whether it is deep or shallow, will influence the possibility of moisture entry into the pavement. Climatic factors such as rainfall, temperature and evaporation may also have an influence.

Surrounding landform including drainage depressions or swamps, adjacent rivers or irrigation areas, vegetation and runoff and permeability characteristics of the nearby soil strata are also important. Finally, the pavement and subgrade material properties such as grain and pore size, density, mineralogy, shrink-swell properties, permeability and occurrence of salinity may also be factors.

Measures to Prevent Excess Moisture

Queensland Department of Main Roads (1993) gave the following information about how to minimise the chance of excess moisture getting into a road pavement.

It was suggested that the works be programmed to minimise exposure to rain, and that stockpiles should be protected to reduce water entry. The amount of time that pavement materials are left uncompacted or in rills out in the open should be kept as short as possible.

When placing the material, the moisture content for compaction should be as low as practicable, and caution should be exercised when doing surface finishing that uses water and backwatering.

Most importantly, it must be ensured an adequate pavement drainage system is designed and constructed, to ensure the pavement is kept as dry as possible during its service life.

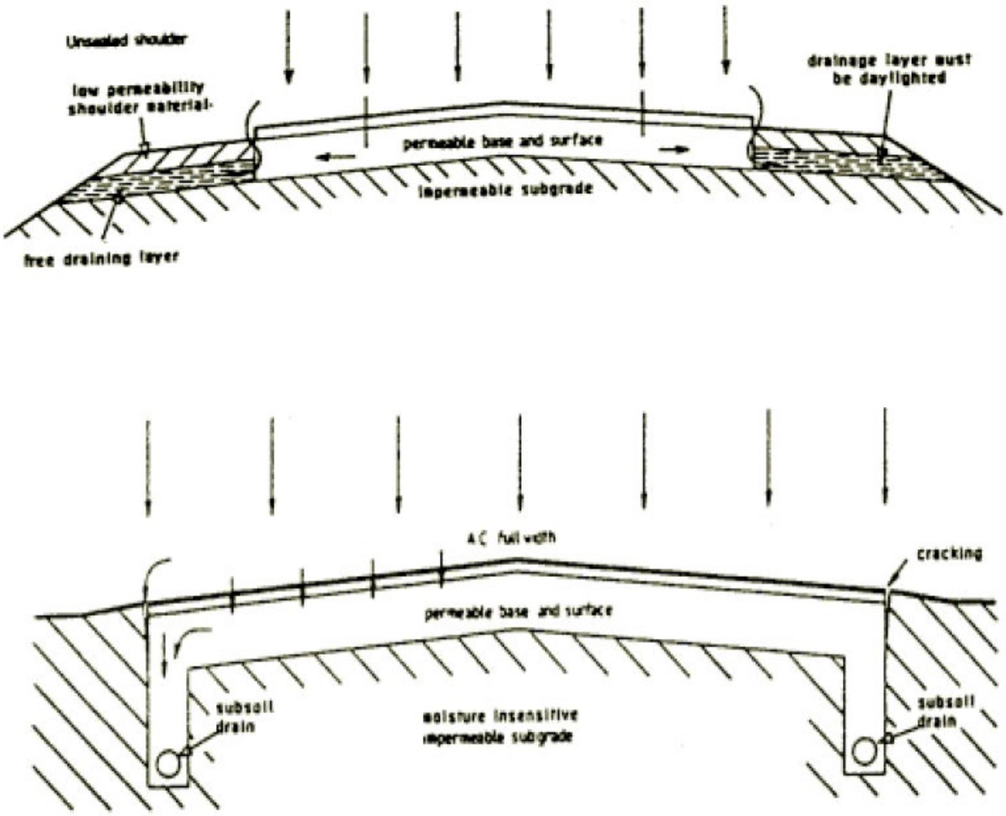


Figure 2.2 – Two alternative drainage systems for a road pavement:
a) Moisture that infiltrates can escape through free draining layers at the side
b) Moisture that infiltrates is channelled to subsoil drains at the sides

2.3.4 Bleeding and Flushing

Bleeding or flushing may also be known as a 'fatty or slick' surface. It is an excess of binder near the surface of the pavement. The binder is either almost covering, or completely covering the surface aggregate.

This results in reduced skid resistance and aesthetics. It also leads to pick-up of binder and aggregate by passing vehicles, leading to reducing waterproofing and higher roughness. The main possible causes are discussed below.

Spraying too much binder is an obvious cause of this failure, but another possible cause is the insufficient application of surface aggregate.

Non-uniformity/patching of the original surfacing as a result of profile correction or remedial maintenance work leads to widely varying surface textures, and this is very difficult to allow for when spraying the binder, the rate of which is dependent on the surface texture.

Any weakness of the base layer of the pavement means that the surfacing aggregate can embed into it, allowing the binder to cover the surface aggregate.

If there is a lack of proper rolling during the placement of the aggregate, this means that the aggregate is not orientated with its least dimension orientated vertically, and when the road is opened to traffic, stripping may occur, leading to excessive binder levels. This will also occur if the newly constructed surface is not protected from traffic for long enough.

Breakdown of the surface aggregate will result in smaller dimensions, allowing binder to cover the aggregate. If there is poor spreading of the surface aggregate, some areas will not be covered adequately, leading to binder levels being too high for this. In an asphalt surfacing, over-filled voids mean that there will be excessive binder, and over time this will rise to the surface.

The rate of binder that should be sprayed is dependent on the size of the aggregate, specifically the average least dimension of the particles. Consequently, if the aggregate has a lack of size compared to that used in the design, this will lead to it being covered with binder.

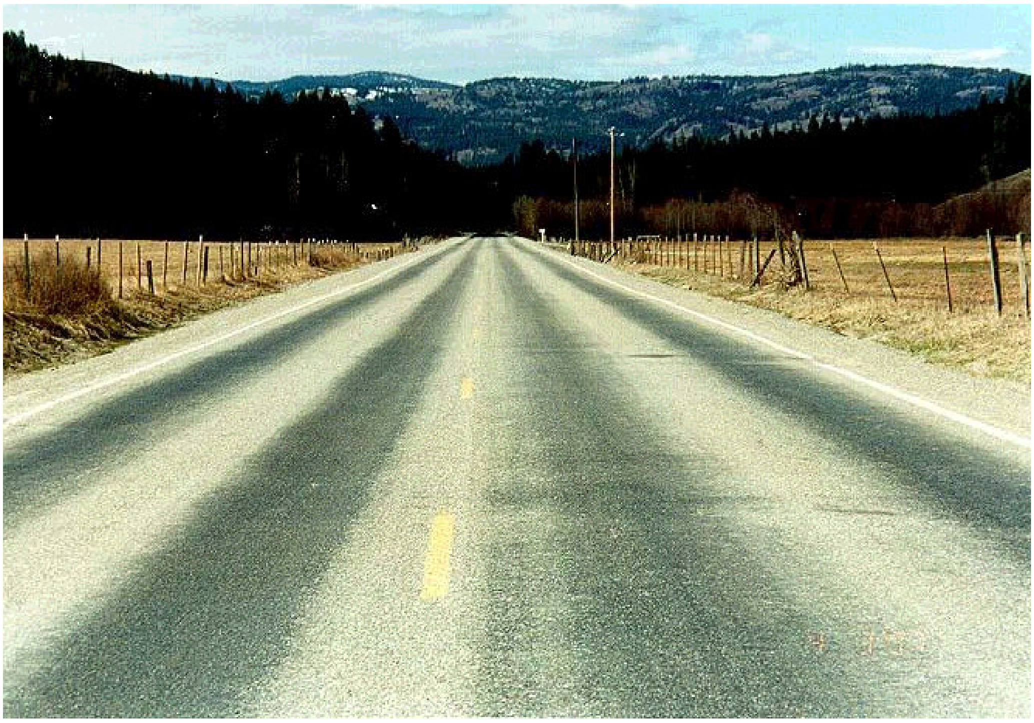


Figure 2.3 – Bleeding and flushing:

a) Within the wheelpaths on a newly completed asphalt surfacing

b) On a rural road

Figure 2.4 – Heavy bleeding in a spray seal

Figure 2.5 – Corrugations on a low volume road in Outback Australia

2.3.5 Corrugations

Corrugations are ripples spaced at regular intervals running transversely across the road. These are mostly found on unsealed roads, and are generally caused by either inadequate stability of the surfacing or base under traffic, or non-uniform compaction of the surfacing or base during construction.

2.3.6 Cracking

Cracking consists of visible discontinuities in surface and can be an indication of the pavement's structural condition and serious, or little more than an aesthetic prob lem (provided cracks are treated). The main problem with cracks is that they allow moisture into pavement, giving accelerated deterioration of pavement.

Cracks can occur in a wide variety of patterns. They may result from a large number of causes, but generally are the result of either ageing and embrittlement of surfacing, environmental conditions, structural or fatigue failure of the pavement, or other causes.

Cracking due to ageing and embrittlement of surfacing

In spray seals, the hardening of binder initially shows as minor aggregate loss, and if untreated leads to cracking and widespread aggregate loss. Further deterioration results in potholes from moisture entry. Cracks in a pavement with an aged surfacing are often of this type.

Environmental Cracking

This type of cracking can result from seasonal moisture or volume change of the subsoil materials leading to shrinkage of the pavement, reflection cracking from underlying jointed pavements, pavement subsidence, crack movement induced by diurnal and/or seasonal changes, or other causes.

Figure 2.6 – Block cracking:

a) In a parking lane

b) Over the entire pavement

Figure 2.7 – Joint reflection cracking on an arterial road

Figure 2.8 – Cold temperatures led to volume change of the pavement materials, and resulted in 'thermal' cracking

Block cracking is where interconnected cracks divide the pavement up into large blocks. It may sometimes occur only in non-traffic areas. It is mainly caused by an inability of the surfacing to expand and contract with daily temperature fluctuations due to either ageing of the binder or a binder that is too stiff.

Reflection cracking is generally caused by vertical and horizontal movements in the pavement beneath overlays that result from expansion and contraction with temperature or moisture changes. Joint reflection cracking results in cracks being reflected up in line with joints in the pavement layer below.

Shrinkage cracking is interconnected cracking forming a series of polygons, usually having sharp angles at the corners. It is usually caused by volume change within the surfacing, the aggregate base, and/or the subgrade layers. If it is caused by temperature changes resulting in volume change, it may be known as thermal cracking.

Cracking due to Structural failure

This generally occurs as closely spaced "crocodile" or "alligator" cracking resulting from fatigue failure of one or more pavement layers. It is generally associated with loss of pavement shape, and pumping of fines may appear on surface due to moisture entry through cracks.

It may reflect permanent severe deformation of the subgrade due to repetitive loading, instability in the upper pavement layers or repeated deflection causing fatigue in the surfacing. This fatigue may also be due to a pavement thickness that is inadequate for the traffic loading. Poor construction practices, including poor compaction, may also be a cause of this type of failure, especially on relatively new jobs.

If surface distortion is also present, excessive moisture is probably the cause. Poor drainage may result in a wet base and/or subgrade. Water may be getting to the base and/or subgrade from cracks or holes in the surface or from moisture coming up through the subgrade. If excessive moisture due to poor drainage is implicated, then this must be corrected.

Figure 2.9 – Fatigue/structural cracking:

a) Due to inadequate edge support

b) On a poorly supported road widening, accompanied by rutting

Figure 2.10 – Severe structural cracking, allowing easy moisture entry into the pavement

Figure 2.11 – Joint cracking due to poor joint location and construction practices

Cracking due to other causes

This includes all other types of cracking that cannot be easily categorised, and the main types are discussed below.

Joint cracking occurs when part of the shoulder or a paved wedge separates from the mainline pavement or along weak seams of adjoining pavement spreads in the surface layers.

This may be caused by wetting or drying action beneath the shoulder surface caused by conditions that trap water and allow it to stand along and seep through the joint between the shoulder and the mainline surface.

Longitudinal cracking occurs parallel to the road centreline. Common causes of longitudinal cracking include poor joint construction or location (e.g. in wheelpaths), a reflective crack from an underlying layer, or structural failure, which may lead to crocodile cracking in the future.

Random cracking has numerous causes that are, in its early stages, difficult to determine. The consequences of these cracks range from severe, such as deep foundation settlement, to slight, such as a construction error or mishap.

Slippage cracks are crescent-shaped cracks that usually point in the direction of traffic movement. They may be the result from insufficient bond between the surface and underlaying courses, caused by dust, oil, rubber, dirt, water, or no tack coat between the two courses.

Transverse cracking occurs at a course approximately right angles to the centre line, usually extending across the full pavement width. Transverse cracks may be the result of reflection cracking, or a result from stresses induced by low-temperature contraction of the pavement, especially if the binder is hard and brittle.

Figure 2.12 – Longitudinal cracking:

a) Possibly caused by ground movement

b) In the wheelpaths, as an onset to structural cracking

Figure 2.13 – Slippage cracking near to a bus stop, caused by acceleration and braking

Figure 2.14 – A depression in the shoulder of a road, probably due to local moisture entry resulting in weakness of the pavement and subgrade

2.3.7 Depressions

Depressions are generally localised within area of pavement, and not confined to trafficked areas. They commonly occur due to settlement of embankments (particularly adjacent to bridge abutments), poorly compacted surface trenches, or due to moisture weakening the pavement.

2.3.8 Edge Defects

Fretting refers to breakage of the surfacing edge and is usually a result of lack of support due to weakening of the pavement – often due to ingress of water or traffic running too close to edge. It may also be caused by inadequate edge support or a weak seal coat with a lack of adhesion to the base.

Drop-off usually results from traffic running off the surfacing and abrading the shoulder material. It is often due to an inadequate lane width or traffic tracking, and is exacerbated by the presence of water. It may also be caused by inadequate edge support or a weak seal coat with a lack of adhesion to the base.

2.3.9 Patching

While not a form of pavement distress, patches are often localised remedial work in response to some form of surface distress. Many recent patches may indicate an ongoing problem requiring overall retreatment. Patches may also be used after utilities have been placed under the road.

Figure 2.15 – A patch placed over an area of localised pavement distress

Figure 2.16 – Polishing of a surfacing. There seems to be a lack of surface texture, but no excess binder.

2.3.10 Polishing

Crushed rock commonly used for surfacing aggregate initially has a rough, skid-resistant texture. Under the action of traffic, however, some aggregates including many limestones become polished and slick, especially when wet. Also, incorrect aggregate selection may result in aggregate that is too smooth and rounded being used.

2.3.11 Potholes

Potholes are a dramatic indication of structural surface failure. Local deflection testing may indicate if problem is due to lack of pavement strength or a problem with surfacing, such as inadequate thickness.

Potholes result from growth of a break in the surfacing, often as a result of severe crocodile (structural) cracking. Once water enters pavement layers, the base and/or subgrade become wet and unstable, and the resultant degradation leads to rapid growth of pothole area and depth.

If the potholes are numerous or frequent, it may indicate underlying problem such as inadequate pavement or aged surfacing requiring rehabilitation/replacement.

Water entering the pavement is often the cause, and could be caused by a cracked surface, high shoulders or pavement depressions ponding water on the pavement, porous or open surface, or clogged side ditches. Any deficiencies should be corrected.

2.3.12 Roughness

Excessive roughness develops when deformation takes place irregularly along pavement. Contributing factors to increase in roughness include the following: depressions, rutting, patches, service trenches, corrugations, potholes, stripping and shoving.

Figure 2.17 – Potholes:

a) As a result of moisture entry due to fatigue cracking of the pavement surface

b) Occurring only in the surface course, with the pavement base still intact

Roughness can highlight variations in materials, conditions and response to traffic loadings, and can be used as an index which provides a good indication of general condition and need for rehabilitation. It is also the main form of pavement distress noted by the travelling public. Generally, to reduce roughness, consideration must be given as to what is causing the roughness and how it can be rectified.

2.3.13 Rutting

Rutting is the permanent downward deformation of the surfacing within wheelpaths. It may result from deformation of the surfacing, the pavement materials or the underlying subgrade, or a combination of these.

It is important to determine which layer is rutting since this will influence the optimal rehabilitation strategy. Generally the layer suffering deformation will be evident from inspection of pits or trenches through the road or other associated indicators. For example, bulging adjacent to the rutting may indicate that the cause is a weak subgrade.

The worse the rutting is, the higher the variation in the transverse profile of road surface. Because of this, ruts interfere with surface run-off patterns and increase the risk of wetting in the upper pavement layers. Rutting can also initiate aquaplaning, and hence have adverse impact on safety.

Deformation in the base layers may lead to reduction of effective pavement thickness, and if untreated, to premature development of deformation in subgrade. It may progress to shoving if rutting becomes so severe that surface cracking occurs, moisture enters and weakens base or subgrade layers.

Rutting may be caused by inadequate strength or stability of either the surfacing, pavement or subgrade, allowing deformation. This inadequate strength may be caused by poor material quality, inadequate pavement thickness or excessive moisture. Loss of strength may also occur due to interaction between layers e.g. cutter in a primer seal penetrating the overlying asphalt.

Figure 2.18 – Rutting:

a) On a road widening, caused by inadequate compaction of the affected area

b) On recently completed asphalt, probably due to instability and poor mix design

If the material is not compacted adequately, further densification under traffic may occur, especially if there is a very high traffic loading of $>10^7$ ESA's. In asphalt, rutting may occur as a result of a poor mix design, with too much binder, too much filler, or insufficient angular aggregate particles.

2.3.14 Shoving

Shoving is the bulging of the pavement surface parallel to direction of traffic, and may especially occur in areas of heavy braking and accelerating such as signalised intersections or steep grades.

Shoving often represents gross deformation of pavement that may rapidly lead to disintegration. If the cause is not immediately evident, trenching may be necessary. Possible failure causes are outlined below.

A cause of shoving is inadequate strength/stability of the surfacing or base material, or a poor bond between the pavement layers, meaning there is no force preventing movement of the layers relative to each other. Lack of containment of pavement edge may also result in this.

An inadequate pavement thickness results in overstressing of the subgrade, and may lead to shoving. Loss of strength and stability may be caused by excessive moisture, contamination caused by oil spillage, or a lack of curing time between placing seal treatments.

In an asphalt layer, shoving may be caused by a mixture that is too rich in asphalt. There may be a lack of resistance to movement, because the asphalt has too much fine aggregate, too much aggregate that is rounded or smooth-textured, or binder that is too soft.

Figure 2.19 – Shoving:

a) Major localised deformation

b) Near an intersection

2.3.15 Stripping and Ravelling

Stripping and ravelling are generally similar failures, except that stripping occurs in spray seals and ravelling occurs in asphalt. They are both mostly caused by a lack of bond between the surface aggregate and the binder, allowing the material to be removed from the surface by passing traffic.

A forerunner of stripping may be a very coarse surface due to insufficient or well-oxidised binder. A variation of ravelling may be known as delamination, which is generally caused by a poor bond with the pavement layer below the surfacing. Main causes of stripping and ravelling are generally similar and are discussed below.

The most obvious cause is inadequate binder application, meaning that the particles can be easily stripped away, due to the resulting low bond between the binder and the aggregate.

A lack of bond between binder and surface aggregate could also be due to a lack of precoating or adhesion agents, or dirty, dusty, or soft aggregate. Degradation of the binder through ageing or other influences (traffic damage, solvent or chemical spillage), or incompatibility between the aggregate and binder or with the previous seal, may also result in a lack of bonding.

Insufficient cutter in the binder during spraying results in not enough wetting of the aggregate by the binder, and results in a poor bond. Conversely, excessive cutter means that the bitumen will often be too soft to adequately grip the particles. These effects may also occur if the cutter is not adequately blended into the binder.

An excessively open graded asphalt mix with many air voids is likely to result in ravelling due to the lack of binder. Poor mix design of any asphalt could also cause this.

Figure 2.20 – Ravelling:

a) Of a 20 year old surfacing, due to ageing and embrittlement of the binder

b) Within the wheelpaths of a road

Figure 2.21 – Stripping:

a) Of a recently completed seal

b) In both wheelpaths on an old seal

During construction, segregation of the aggregate and the resulting variation in size may result in too large an aggregate size in one location, allowing easy stripping. Inadequate compaction, a lack of rolling, or excessive brooming means that the aggregate particles do not align with their least dimension vertical, allowing them to be torn out by traffic.

If the surface aggregate is fracturing or breaking down, parts of it can be removed. Some aggregates are hydrophilic, meaning they tend not to bond well with water, and could result in a poor bond with the binder.

A dry brittle surface prior to sealing may mean that there will be a poor bond with the new seal, allowing easy stripping. Patching work carried out deeper than the base material may mean a poor bond with the next layer down.

Excessive heating of the binder or asphalt during mixing reduces the effectiveness of the bitumen to grip the aggregate particles when sprayed, and there will normally be a poor bond during wet or cold weather due to moisture or stiffening of the bitumen.

2.4 Pavement Testing

2.4.1 Introduction

Pavement testing aims to provide information about the behaviour of the pavement. This information may be used to determine the cause of a pavement failure, and to determine the best rehabilitation treatment.

Testing can be either destructive or non destructive, depending on whether the pavement structure is disturbed or not. Before these types of testing are summarised, parameters that are often considered when evaluating a pavement are discussed. Further details about how these properties are tested are discussed in the following chapter, where the investigation method is developed.

2.4.2 Road Condition Parameters

Austroads (2000a) states that road condition parameters may be considered to be in two categories as discussed below. The distinction between these two categories is not precise, and the majority of pavement failures would influence factors from both of these categories.

Road user and community parameters are those that are most often noticed by the road user, even if there is no structural failure of the pavement occurring. The main parameters that are often considered are as follows:

- Pavement surface shape (including roughness)
- Skid resistance
- Noise
- Rutting and shape loss
- Visibility of markings and reflectivity
- Appearance
- Water spray

Structural parameters are those measures that may indicate a problem with the structural capacity of the pavement, which may lead to further deterioration in the future. The main parameters that are often considered are pavement strength, cracking and serviceability, particularly ageing effects.

For the purposes of this project, the parameters from the above lists that are considered most important are as follows:

- Pavement surface shape (including roughness)
- Skid resistance
- Rutting and shape loss
- Pavement strength
- Cracking

Queensland Department of Main Roads (2002c) states that the above factors may be reduced to three general road condition measures, which are:

- Pavement shape, including the effects of roughness and rutting.

- Pavement surface integrity, including cracking, surface texture, and skid resistance.

- Pavement strength, which may be assessed using materials sampling and testing, or by measuring the pavement deflections.

Austroads (2000a) gives an introduction to the assessment of these measures, a summary of which is as discussed below.

Pavement Shape

Pavement shape and its deviation from design values may be considered by measuring either the longitudinal road profile, which is typically roughness related, or local shape variability, which includes the transverse road profile.

Roughness is a general measure of the deviation of the longitudinal road profile from the design. It is often the main form of road condition noted by the travelling public, and should therefore be considered important. Common measures of Roughness in Australia are NAASRA (Austroads) Roughness Counts and the International Roughness Index.

Instruments used to measure roughness in Australia include the NAASRA roughness meter (which is a sensor mounted on the rear axle of a standard vehicle), laser profilers, or ARRB TR walking profilers. The roughness meter is cheap and simple, suited best for low cost applications. Laser profilers fitted on a special vehicle are commonly used to give roughness measurements over a long length of road network, whereas the walking profiler is used for short road sections.

Another measure of road roughness may be subjective measurement while driving over the road section. While the roughness experienced and recorded is heavily dependent on the vehicle type and the operator's consistency, it may be a useful measure. Similarly, if members of the public consistently complain that a road section is too rough, then this may well be the case.

Local shape variability is any variation in the road profile away from acceptable values. Variation in the transverse profile could be caused by rutting or edge drop-off, while other variations may be caused by corrugations, depressions, shoving, surface irregularities, or a lack of local smoothness.

Shape variability is usually measured as deviation from 1.2m or 3m straight edges and measured in mm. Laser profilers can also assess transverse profile of the road; with slightly varying methods used in different states of Australia (see Dowling, 1998).

Pavement Surface Integrity

The function of a pavement surfacing is to provide a skid resistant surface for vehicles, and to prevent water entry into the pavement through the surface. The effectiveness of the surfacing in achieving these aims can be assessed by using the measures that are discussed below.

Skid resistance is a measure of how well the road surface allows vehicles to brake without skidding, in both wet and dry weather. This may be measured using the following basic types of friction measurement:

- Locked wheel testing, which uses a smooth tyre momentarily locked on the road surface.

- Fixed slip testing, where a standard test tyre rotates at a constant percentage of slip.

- Sideways force coefficient measurement, that be assessed on a network level using a SCRIM machine. This is the most common method used in Australia.

- Variable slip testing, of which the Norsemeter machine is an example.

- Portable pendulum skid resistance testing. This device uses a pendulum that is manually operated, and is suitable for only small areas.

The data provided by this testing may provide an indication of the possible danger to vehicles, if sudden braking is required. The importance of skid resistance is therefore greater in areas where braking is likely to occur.

Surface texture provides an indication of the volume through which water may escape from the road-tyre interface, and is therefore related somewhat to skid resistance.

However, Dowling (1998) has stated that it should be noted that surface texture data is rarely used, except for planning surface treatments. Surface texture is commonly measured using sand-patch test. In addition, it may also be measured using visual assessment or laser profilers.

Cracking allows moisture entry into the pavement through the surface, and so is undesirable. Dowling (1998) states that it may be measured using continuous observation by a person in a vehicle, a detailed visual examination of sample areas, or an automated method as utilised in a network-level survey machine. Important information to be recorded would include the extent, type and width of cracking that is occurring.

Pavement Strength

Pavement strength is very important since it provides an indication of how the pavement will behave structurally, when loading under traffic occurs. Local shape variability and a lack of pavement surface integrity may be corrected, but a lack of pavement strength will probably be much more expensive to correct.

Testing of pavement strength may be divided into non-destructive and destructive testing. Testing involves the use of special equipment and may be quite expensive, especially if traffic control and repair of the pavement structure is required.

2.4.3 Non-Destructive Testing

Deflection Testing is the most common form of non-destructive testing carried out on a road pavement. It assesses the strength of a pavement by indirect means, by measuring the response of the pavement to a loading. While the results are sometimes open to interpretation, it still may be a valuable form of testing to use.

When using deflection testing, there may be a need to convert deflections using one method to the equivalent deflections using another method. Guidance on this matter is provided in *Pavement Strength in Network Analysis of Sealed Granular Roads: Basis for Austroads Guidelines* (Austroads, 2003a).

More detailed information about this form of testing is included in the following chapter, where the investigation method is developed. In addition, other forms of non-destructive testing will be discussed.

2.4.4 Destructive Testing

Mooney et al. (2000) has stated that destructive testing is often necessary to determine the true cause of a pavement failure, since only so much can be seen from non-destructive testing, such as deflection testing.

Destructive testing must often be done using either trenching or coring to obtain samples. Subsurface profiles may be taken to see deformation of different layers, and to check that recorded layer thickness profiles are correct (Chen et al., 2003).

Trenching also provides a visual view of the pavement layers, and an assessment can be made of the wetness of each layer, and any moisture at interfaces between them (Mooney et al., 2000). It also can give an idea of the bond between the layers.

Specific testing methods available for use in Queensland are listed in the *Materials Testing Manual* (Queensland Department of Main Roads, 2002a).

Other information about testing available in Australia can be seen from the AS1289 List of Testing Methods (Standards Australia, 2002), the Roads and Traffic Authority of New South Wales (2003), Vic Roads (2004b) and Main Roads WA (2004).

These tests are currently mostly empirical testing methods, and as such the procedure used to carry out the test must be consistent to ensure results are not distorted.

The use of Repeat Load Triaxial (RLT) testing and other less empirically-based testing methods will be expected to increase in the future, due to its benefits for the characterisation of the behaviour of pavement materials. Further information about this can be found in *Development of Performance-based Specifications for Unbound Granular Materials* (Austroads: 2003b and 2003c).

The general types of tests currently used by road agencies in Australia will be discussed below. More detailed information about these tests can be found in the following chapter where the investigation method is developed.

Soil and Aggregate Tests

The California Bearing Ratio (CBR) test is a commonly used empirical test used to estimate the strength of the soil. Repeat Load Triaxial (RLT) testing is more advanced and gives a better indication of material behaviour, and its use will be expected to increase in the future.

In-situ strength measurement may be completed, often using a dynamic cone penetrometer (DCP). Other types of tests may be used to assess the resistance of the aggregate to polishing or crushing, or to assess the soundness of the material when exposed to chemical attack.

Moisture content measurement can provide an indication of whether there is excess moisture in the road pavement. The degree of saturation can also be calculated, and this measure has more of an influence on the behaviour of the material and how it is likely to fail.

The particle size distribution or grading of the material can be assessed, typically using sieves, to determine the percentage of material within particular sizes. Various other tests may be used to assess the particle size and shape, such as the flakiness index and the average least dimension (ALD), which is an important parameter for spray seal design.

The dry density of a soil sample can be assessed by obtaining an in-situ sample, typically using sand replacement, drying this out and calculating the dry density (moisture content can also be found). Testing a number of samples enables the construction of the dry density – moisture content relationship.

Other material properties can be assessed, using the Atterberg Limits tests, linear shrinkage test, or other tests as needed.

Geotechnical Tests

Geotechnical tests may be conducted to assess the soil or rock strength using penetration devices, measure the water table level, or determine the compressive strength of the material.

Other tests may also be carried out, such as triaxial or shear strength tests to assess material strength characteristics, or consolidation testing to measure the materials deformation behaviour under an applied load.

Asphalt Tests

Asphalts tests may be used to assess the strength and durability of the asphalt, or other properties. Varying types of tests are used in the mix design of asphalt, and also to assess the deformation properties of the asphalt.

Bituminous Material Tests

Bituminous material properties may be assessed using many types of testing, including those for general bitumen products, and those for polymer modified binder. Tests used may include viscosity or penetration testing.

2.5 Pavement Failure Investigation Methodologies

2.5.1 Introduction

This section will examine methodologies that have previously been developed for the investigation of pavement failures, both overseas and in Australia. The steps in each methodology will be compared to find common steps that could be utilised in the methodology to be developed and utilised for this project.

2.5.2 Overseas

Generally, the methodologies developed overseas have been found mostly in Texas and the USA. This may be because these are the only published reports available on the Internet.

Basic concepts, current practices, and available resources for forensic investigations on pavements (Victorine et al., 1997)

This reference lists the general steps in a forensic investigation of a failed pavement as used by Texas Department of Transportation, and these steps are shown below.

First there is a preliminary meeting between the coordinator of the project and the investigative team, the purpose of which is to review the facts of the case and become familiar with the local area and the project.

Next there are interviews with people familiar with project, such as the construction engineer, the project engineer and inspector, and the laboratory or materials testing supervisor.

An onsite investigation is conducted, to evaluate the condition of the existing pavement. A visual examination allows the set up of an effective testing procedure, and provides additional support in developing alternate rehabilitation strategies.

The project records are reviewed, taking note of details such as road sections, history, materials, and type of failure. Also reviewed are any soil, geological, traffic, temperature, and weather (rainfall) data that may be relevant.

A detailed condition survey is carried out, the first part of which is detailed visual evaluation of the failure. Deflection testing may be carried out using the falling weight deflectometer (FWD). Testing may also be carried out using the dynamic cone penetrometer (DCP), ground penetrating radar (GPR), or a TxDOT profilometer.

The presence and location of the water table is determined, as well as the drainage condition on the road. Other types of testing that may be used include the spectral analysis of surface waves and the multi-functional vehicle.

Materials sampling and laboratory testing is carried out. Coring may be used to determine layer thicknesses, and the bonding and integrity of layers. Trenching is used to get samples of the pavement layers for testing, and may also be used to examine the thickness and condition of layers, and help to determine which layer(s) are rutting. This is generally used only in critical situations.

Subsurface investigations of the underlying soil may be used to identify and classify subgrade materials, based on the soil type and the types of stabilising materials present, such as cements. Other testing may be conducted to determine the presence of sulfates, the triaxial compressive strength, the plasticity index (PI) and the liquid limit.

The data gathered in the above stages is analysed to help identify the most likely cause(s) of the problem. This process is often ongoing throughout the project, and uncertainties often remain. However, the most likely cause(s) of the problem must be determined, and these are typically a result of construction, materials, design or environment related factors.

A final report is produced that documents in detail the entire forensic investigation. This should include detail about the project history and background (if the failure occurred on a specific job), pavement structure and material types, failure modes, testing and investigation carried out, and the conclusions about the cause(s) of the failure, with a prioritised list of possible corrective strategies and costs

Development Of A Formal Forensic Investigation Procedure For Pavements (Crampton et al., 2001)

This reference details a forensic investigation methodology specifically developed for use in Texas, USA, which includes the steps shown below.

Firstly, background information is gathered. This might include details about previous condition surveys, relevant documents, pavement history, pavement structure, materials information, traffic information, a description of the distress, and relevant construction, weather, soil and geological records.

Next, it is usual to prepare for the investigation. This will involve planning the investigation, identifying the investigative team, set up an operations planning framework and conducting a literature search.

On the initial site visit, field observation and documentation should be used to formulate possible failure hypotheses, and determine testing required, both non-destructive and destructive, that will be carried out.

When all testing has been completed, the failure hypotheses developed should be tested by analysing the collected data and information. To do this, it is necessary to organise the information, and analyse data. This may be done using a matrix method listing known information against the failure hypotheses, and determining how well these match.

The conclusions about the failure should be formulated, such as the probable failure cause(s), and from this the best rehabilitation options. To conclude the investigation, a forensic report is prepared, containing details about the investigation purpose, road history, a description of project, observations of the failure, any testing conducted, the analysis of the data, and conclusions and recommendations.

2.5.3 In Australia

In Australia, several methodologies have also been developed for the investigation of pavement failures. The steps in each will be discussed below.

Pavement Rehabilitation Manual (Queensland Transport, 1992)

This focuses on the investigation of pavements for rehabilitation and has the following main steps shown below.

Firstly, the need and purpose for rehabilitation work should be established, by considering whether the road section is still safe for users, and if widening or alignment work may also be required.

The pavement should be evaluated. Initially any historical data related to design, construction or maintenance should be reviewed, as well as any data about climate or traffic.

The functional condition of the pavement should be examined, by considering how well the pavement performs its intended function. This should be done by considering roughness, geometric form, noise, skid resistance and other relevant parameters.

The structural condition or capacity of the pavement should be examined. This is firstly done by examining the road surfacing and any defects in it. Next the structural response to load can be assessed using deflection testing. Material properties may also be assessed using other non-destructive or destructive testing methods.

The moisture control system of the pavement should be examined for defects that may be contributing to the failure. It is also important to have considered the moisture environment, and the effects that moisture may have on the pavement structure.

Once the pavement has been evaluated, alternative rehabilitation options can be selected by considering economic factors, as well as design and construction considerations. Once the specific rehabilitation treatment has been selected, this can be designed using guidance provided in the manual. Treatments may include fixing of the moisture control system, surface treatments, asphalt or granular overlays, in-situ stabilisation, or another form of treatment.

Site investigations for the rehabilitation of low-trafficked road using in-situ recycling (AusStab, 2000)

Firstly, a desktop evaluation of the pavement should be conducted. This should involve the review of the pavement composition, traffic loading, current performance data, and original construction plans. In addition, related employees should be interviewed, and pavement management systems may be able to provide important information.

A site evaluation should be carried out. The condition of the pavement should be examined, and proof rolling might be used to provide an indication of any structural inadequacy. Drainage patterns and existing drainage structures should be noted, and the effectiveness and maintenance of these should be examined. Topography and geology, any other important objects, and an indication of the traffic volume and composition should also be recorded.

An evaluation of the pavement materials should be carried out. The pavement surface condition should be examined. Samples of material may be taken for laboratory testing, such as plasticity index and sieve analysis, and CBR testing.

Trenching may be used to examine the thickness and composition of each layer, and allow visual assessment of the density and moisture content of layers. In-situ CBR testing may be conducted using a DCP (dynamic cone penetrometer), with a depth of at least 500 mm below the top of subgrade being recommended for testing.

GeoPave Technical Note 66 – Guide for Pavement Investigations (VicRoads, 2004a)

This has the following main steps, for the investigation of pavements for rehabilitation, shown below.

Firstly a desktop investigation should be carried out, where the findings of any previous investigations, and possible constraints on the rehabilitation are recorded.

A visual inspection allows the condition of the existing pavement to be examined, as well as the general site conditions. This stage should be used to get a detailed record of any pavement distress, and a more detailed examination of any possible constraints on rehabilitation treatments.

When examining the subgrade material, it is important to note the soil type, material density, moisture contents and plasticity.

Pavement materials investigations may be conducted by the excavation of a small area to determine materials types and thicknesses, and provide samples for later laboratory testing. In-situ strength testing or coring may also be used. The strength of the material may be assessed using deflection testing.

The surface of the pavement may be investigated using deflection testing, skid resistance testing, ground penetrating radar, or a surface condition rating system.

2.5.4 Comparison of Methodologies

The methodologies listed above generally contain similar steps, with some variation in the terminology used and the exact order in which they are carried out. Some of the publications do not include some steps, depending on the detail of the publication and the investigation purpose.

From considering the information, eight steps can be selected for the investigation method to be developed in this project, as follows:

- Plan the Investigation
- Review Documents and Literature
- Interview Personnel
- Non-destructive condition survey
- Destructive materials sampling and testing
- Determine probable cause(s) of failure
- Determine best rehabilitation treatment
- Report on outcomes

How those steps related to the methodologies previously developed is shown in the table on the following pages. It can be seen that most methodologies contain steps that may be listed as part of one of the steps listed above.

Table 2.3 – Comparison of pavement investigation methodologies: Australia and overseas

Victorine et al. (1997)	**Crampton et al. (2001)**	**Queensland Transport (1992)**	**AusStab (2000)**	**VicRoads (2004a)**
1) Plan the Investigation				
Preliminary Meeting	Prepare for investigation,	Identify need and purpose of rehabilitation		
2) Review Documents and Literature				
Review of project and other records	Background information gathered	Historical and other data reviewed	Desktop evaluation	Desktop investigation
3) Interview Personnel				
Interviews with people familiar with project or road			Interview related employees	
4) Non-Destructive Condition Survey				
Visual investigation, detailed condition survey	Initial site visit and non-destructive testing	Examine functional, moisture and structural condition	Site evaluation	Visual inspection and deflection testing

Victorine et al. (1997)	**Crampton et al. (2001)**	**Queensland Transport (1992)**	**AusStab (2000)**	**VicRoads (2004a)**
5) Destructive Materials Sampling and Testing				
Materials sampling and testing, subsurface investigation	Destructive testing	Materials testing	Examination of pavement materials	Pavement materials investigations
6) Determine Probable Cause(s) of Failure				
Analyse data to find likely failure cause	Analyse collected data in a systematic way			
7) Determine Best Rehabilitation Treatment				
Included in final report		Select alternative options for analysis, decide on treatment		
8) Report on Outcomes				
Final report to detail entire forensic investigation	Forensic report prepared			

2.6 Pavement Failure Investigation Case Studies

This section lists pavement failure investigation case studies that were found during the literature review. Many other sources may also contain case studies that could be useful when examining a similar pavement failure.

The document *Basic concepts, current practices, and available resources for forensic investigations on pavements* (Victorine et al., 1997) cites case studies from other sources include the following:

- The post-construction failure of a lightly trafficked road in Ghana
- The failure of the cement-treated base on State Highway 36, Houston, Texas
- The use of microsurfacing in highway pavements in Texas, and problems
- Further flexible pavement case studies in Texas

The document *Development Of A Formal Forensic Investigation Procedure For Pavements* (Crampton et al., 2001) has details of several case studies including the following:

- Odessa District, State Highway 158, Texas
- Houston, State Highway 36, Texas
- A concrete pavement at a military airfield

Other publications that include information on pavement investigations include the following:

- *Forensic Evaluation of Premature Failures of Texas Specific Pavement Study – 1 Sections* (Chen et al., 2003)

- *Forensic Investigation of Pavement Distortions using Soil Suction* (Park et al., 1999)

- *Importance of Invasive Measures in Assessment of Existing Pavements* (Mooney et al., 2000)

- *Forensic Study of Warranty Project on US82* (Chen et al., 2002)

- *DCP Criteria for Performance Evaluation of Pavement Layers* (Gabr et al., 2000)

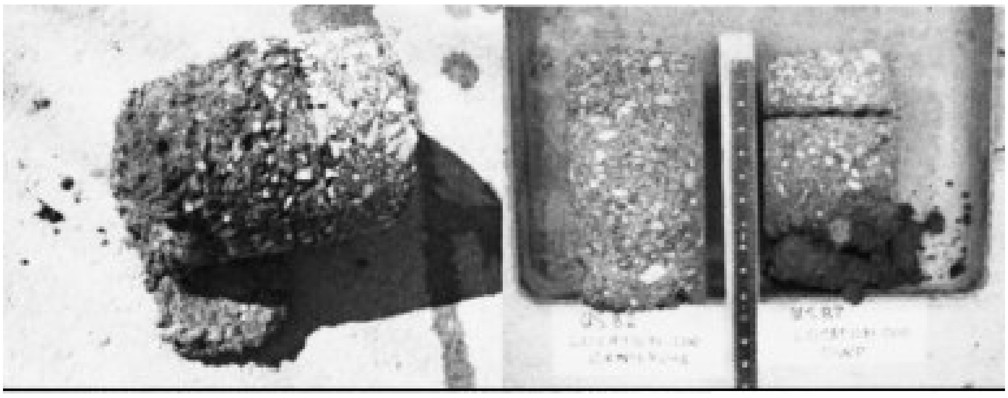


Figure 2.22 – A deteriorating pavement core, an intact core from the road centre, and a deteriorating core from the outer wheelpath

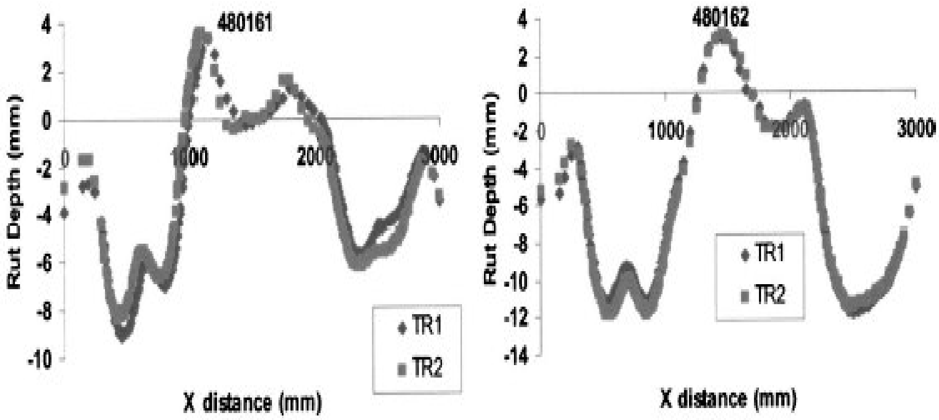


Figure 2.23 – Transverse surface profiles as measured for different road sections

Chapter 3

The Investigation Method

3.1 Introduction

In this chapter, the investigation method is developed. This was achieved by utilising the most relevant information contained in the sources mentioned in the previous chapters, and with variations to ensure that the method is flexible enough to be used for most types of pavement failure, and in a variety of locations.

3.2 Investigation Method Outline

The systematic investigation method has four basic steps as follows:

- Plan the Investigation
- Retrieve all possible data required for the problem
- Analyse the data obtained
- Conclusion

These can be further categorised as follows:

- Plan the Investigation
- Review Documents and Literature
- Interview Personnel
- Non-destructive Condition Survey
- Destructive Materials Sampling and Testing
- Determine probable cause(s) of failure

- Determine best rehabilitation treatment
- Report on outcome of investigation

It should be noted that often these steps are not carried out in sequence, but instead may occur simultaneously or at varying times, as depending on the circumstance of the project. Also, some steps may not be carried out at all, depending on the magnitude of the failure and the resources available. A more detailed summary of each of the steps in the investigation procedure is given below.

3.2.1 Plan the Investigation

A general review of the problem should first be conducted. The possible scope of investigation and rehabilitation work that may need to be carried out can be considered by examining the importance of the road, the funds available for investigation and rehabilitation, the magnitude of the failure, the current risk to road users, the risk of further deterioration in the future, and any planned future work.

Next, an investigation plan should be drafted, although this may well change as the investigation progresses. This plan should address goal setting, budgeting constraints, operations planning and the investigative synthesis. In addition, the investigation team should be decided upon.

3.2.2 Review Documents and Literature

This may involve the inspection of plans, the pavement history, the drainage design, pavement materials information and specifications, and previous material test results.

Other information that should be examined includes construction records, testing methods and frequencies, and other relevant information, such as traffic volumes and composition, soil or geological records, and temperature, weather or rainfall data.

Another potential source of information may be publications, either from within the road agency, or from other sources, both in Australia and overseas.

3.2.3 Interview Personnel

Personnel that may need to be interviewed may include designers, construction or maintenance personnel, and any other personnel that may have useful information.

3.2.4 Non-destructive Condition Survey

Firstly a visual examination of the pavement should be carried out. The pavement failure should be examined in detail, and the effectiveness of drainage structures and the presence of moisture should also be looked at. Salinity and rising water tables may be a factor.

Road details such as topography and alignment should be recorded, and the soil and geology of the surrounding area may also be of importance in determining the cause(s) of the pavement failure.

Deflection testing of the pavement may be carried out, ranging from network level testing over a long length of road to more detailed testing of specific road sections using the Benkelman beam or Falling Weight Deflectometer (FWD). The interpretation of the information gained from deflection testing is somewhat difficult, but useful information can be gathered.

Other forms of non-destructive testing may also be carried out, including seismic surveys, roughness and surface evenness measurement, skid resistance testing, defect surveying and mapping, and Ground Penetrating Radar (GPR) testing.

3.2.5 Destructive Materials Sampling and Testing

Destructive materials sampling and testing may be carried out on the pavement failure if it is deemed to be necessary in providing more information about the cause(s) of the pavement failure and possible rehabilitation treatments.

Trenching or coring may be used to provide material samples for laboratory testing, and also allows a visual examination of the pavement structure.

Destructive testing may include tests on the soils and aggregate, geotechnical tests, tests on asphalt and tests on bituminous materials.

Tests on soils and aggregates may aim to assess the strength and durability of the sample, the moisture present, the particle size and shape, the material density, or other properties such as plasticity. Geotechnical tests may include measurement of the water table, the compressive strength of the sample, or other material properties such as cohesion or shear strength.

Asphalt tests may be used to assess the strength and durability of the sample, through asphalt mix design, or measuring the deformation behaviour, and other properties may also be tested. Bituminous material tests are categorised into those tests carried out on normal bituminous materials and those carried out on polymer-modified binders. Common tests include measurement of viscosity and penetration of the sample.

Test methods used in the Queensland Department of Main Roads Standard Specifications that are relevant for pavements are listed.

3.2.6 Determine Probable Cause(s) of Failure

It is normally impossible to say with complete certainty what the cause(s) of the pavement failure being investigated was. Instead, the probable cause(s) are normally stated, and there are often multiple factors that contributed to the failure.

The fist stage in determining the failure cause(s) is the investigative synthesis, where all the information gathered is recorded in a logical manner, typically in a report format.

From this listed information, it is then necessary to determine which information supports or refutes each of the possible failure hypotheses. This may be initially be done by considering general failure causes, such as those related to construction, materials, design, or the environment.

It is more usually required that specific cause(s) of the failure be considered. This is achieved by going through possible failure cause(s) for the failure type and determining whether there is any evidence to suggest that this was a factor that contributed to the failure.

Once this has been done, it is necessary to determine the probable cause(s) of the failure. This can be done in a systematic way using a matrix method where the facts are listed against the failure hypothesises, enabling a comparison of how well these match to be conducted. Once the probable failure cause(s) has been revealed, information that has been learnt from the failure can be listed, for inclusion in the final report, to help prevent similar failures in the future.

3.2.7 Determine Best Rehabilitation Treatment

When selecting the best rehabilitation treatment, it is first necessary to list a variety of alternatives that may be feasible, from an initial examination of the conditions. These possible alternatives can then be subjected to a much more detailed examination.

The first part of the detailed examination is an economic comparison of the possible rehabilitation options, considering costs associated with construction, maintenance, salvage value, and road users.

Other factors should also be considered, such as the effect on the public, grade line and geometry restrictions, stage construction, traffic management, risk, design sensitivity, construction tolerances, and the availability of plant and material.

Rehabilitation options may be split up into those for granular pavements, asphalt pavements, and the rehabilitation of the moisture control or drainage system.

Treatments may include surface treatments, overlays, in-situ stabilisation, or other miscellaneous rehabilitation treatments. These treatments can be designed with guidance from the *Pavement Rehabilitation Manual* (Queensland Transport, 1992) or other sources as appropriate.

3.2.8 Report on Outcome of Investigation

A report on the outcome of the pavement failure investigation should be produced, as this enables others to learn from the failure, and should help reduce the chances of a similar failure in the future. Information that should be included includes a general review of the project and its location, failure details, a description of any testing carried out, what the probable cause(s) of failure was deemed to be, how it could be prevented in the future, and possible rehabilitation options.

3.3 Plan the Investigation

3.3.1 Introduction

Planning is important to ensure that the investigation of pavement failures carries out its intended task within a reasonable time frame, and at the lowest reasonable cost.

It is important to know from the onset whether the purpose of the investigation is to determine the cause of the failure, determine the best rehabilitation treatment, or as in most cases, to do both these things.

While the investigation plan may often be modified as the investigation proceeds, the initial plan provides a good basis for ensuring that the investigation achieves its objectives.

3.3.2 General Review of the Problem

Before any planning can be done it is necessary to firstly do a general review of the history of the road, the project, the failure occurring and the location. This information need not be detailed, since this information will be covered in more detail in later steps of the investigation.

The information gathered serves as a guide as to how the investigation should proceed. Road history may provide evidence of previous similar failures, and details about the project are generally required (if the failure has occurred on a newly constructed project).

It is always necessary to have a general idea of the type and magnitude of the pavement failures, and the location where these are occurring may provide a tentative indication of the cause. However, past experience should not be relied on too early in the investigation, since this may mean that other probable causes of the failure are not fully considered.

The information listed here can generally be gathered from plans, on-site investigation, or speaking to knowledgeable people such as the local area inspector, or whoever brought the failures to attention.

3.3.3 Scope of Work

When planning an investigation of pavement failure, in the initial stages, it is always important to have general idea of the likely scope of the work. This may change with varying circumstances, but a preliminary estimate of this is always helpful. The main factors influencing this are listed below.

Importance of the road

This is dependent on several factors including road class, which in Queensland, Australia may be National Highway, State Strategic, Regional, District, or a non state-controlled road, maintained by the local council.

Generally, the higher the road class, the more important the road, although this depends on the location where the failures are occurring, and other factors including:

- AADT (Average Annual Daily Traffic)
- HV (Heavy Vehicles/day)

Generally the higher the AADT and Commercial Vehicles, the more important the road, since this means more people and freight-carriers are using the road, and likely to be effected by the failures. Consideration should also be given as to whether the road section where the failure is occurring is an isolated link, or could be bypassed if necessary.

Funds available

This is often related to the importance of the road, but also varies depending on the current location within the budget cycle, and the available budget. The source of funds may vary depending on the road type.

For example, the Federal Government funds National Highways, and the State Government funds the other state-controlled road classes. Local council roads are funded from another source.

The funds to be considered include both:

- Funds available for investigation of the failure
- Funds available for rehabilitation work

A detailed investigation cannot be carried out if the funds are not available for this. In addition, there is no point carrying out a detailed investigation, if there are only funds available for the cheapest rehabilitation option, whether this is best option over the road life cycle or not.

Magnitude of the failure

The magnitude of the failure and its progression can often be measured if the failure is fairly gradual, using information from asset management data programs. For example, information about State-Controlled Main Roads in Queensland is available from the ARMIS (A Road Management Information System) using the Chartview program. In the case of a rapid failure, however, it is more difficult to determine. The main measurements of the magnitude of the failure are discussed below:

Roughness

The magnitude of the failure is often difficult to define, but the main failure of a road noticed by the general public is excessive roughness. This not only causes discomfort, but also can put considerable wear and tear on vehicles. Heavy vehicles are normally more adversely affected by roughness than light vehicles.

For these reasons, the roughness of the road should always be considered. Roughness can be measured using a specially equipped test vehicle that can give exact values (and also costs money), or by an indicative qualitative indication of the roughness. Another guide is the number of complaints received about the road section.

Guidelines as to the desirable and actual roughness levels of roads at retirement (or significant rehabilitation) are suggested in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992), listed in the table.

Table 3.1 – Typical roughness levels at road retirement in NAASRA counts

Road Type	**Roughness at Retirement**	
	Desirable	**Actual**
Urban Arterials, Sub-Arterials and Highways	110	200
Main Roads, Developmental Roads and Secondary Roads	175	230

However, these values are only a guide. In practice, acceptable roughness values are likely to be based on the condition of the local road network, and the typical roughness values for roads carrying similar traffic volumes.

Excessive Maintenance Costs

This is more likely to be a deciding factor on relatively low volume roads where the failure is slowly occurring. However, it may also be a factor on other roads, and to determine whether this is important, a cost benefit analysis could be performed, comparing the cost of the road using current practice with that after some rehabilitation work has been done.

Magnitude of Deformation

This refers to failures where deformation is occurring, such as rutting or depressions. The exact level of deformation can be measured using a specially equipped vehicle (that can give exact values and various statistical measures) or more conveniently and cheaply, using manual methods, such as a straightedge and level. Suggested terminal rutting values are suggested in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992), shown in the table.

Table 3.2 - Terminal depth of rutting required for rehabilitation

Road Type	Terminal Rut Depth (mm)	Length of Road exceeding terminal rut depth (%)
Heavily Trafficked (>2000 vehicles/day)	20	20
Lightly Trafficked (<2000 vehicles/day)	30	20

Area of Road affected

This is suitable for most failures, with the condition that the percentage area of road affected within the section should be measured as a percentage of the area of the wheelpaths or the total road area, depending on the failure location.

Length of Road affected

This is perhaps the simplest measure of a failure. Suggested guidelines for crocodile (structural/fatigue cracking) are shown in the table, from the *Pavement Rehabilitation Manual* (Queensland Transport, 1992).

Table 3.3 - Terminal extent of crocodile cracking recommended

Road Type	Length of Road Cracked (%)
Urban Arterial, Sub-Arterial	5
Highways, Main Roads and Developmental Roads	20
Secondary Roads	30

Location of the Failure

The magnitude of the failure is dependent on where on the road the failure is occurring. Obviously a failure in the wheelpaths (where vehicles travel) is likely to be worse than a failure away from the traffic location.

Loss of Skid Resistance

This refers to failures where skid resistance is likely to be reduced, such as bleeding or flushing, and polishing of aggregate. Skid Resistance may be measured using a portable pendulum tester (suitable for small areas), a SCRIM machine that measures Sideways Force Coefficients (SFC) over a large area, or some other form of testing. Suggested guidelines are suggested in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992), listed in the tables below.

Table 3.4 – Recommended wet skid resistance values using portable pendulum tester

Category	Description	Minimum Skid Resistance Value (Wet)
A	Roundabouts, Sharp bends, Intersections, Steep Gradients, Approaches to lights or other sites with unusual geometry	65
B	Motorways, trunk and class 1 roads in urban areas (>2000 vehicles/day)	55
C	All other sites	45

Table 3.5 – Recommended minimum sideways force coefficients (SFC) using SCRIM, at 50 km/h

Site	Definition	Risk Rating		SFC	
		Min.	**Max.**	**Min.**	**Max.**
A1 (Very Difficult)	Approach to signals where speed > 64 km/h; approach to signals, crossings and hazards on main urban roads	6	10	0.55	0.75
A2 (Difficult)	Approach to major junctions, carrying >250 C.V. / lane-day; roundabouts and approaches; sharp bends or steep gradients	4	8	0.45	0.65
B (Average)	Straight sections or large curves on motorways, principal roads, and other carrying >250 C.V./day	1	6	0.30	0.55
C (Easy)	Straight sections on lightly trafficked roads; other locations where accidents unlikely	1	4	0.30	0.45

Current risk to road users

This is strongly related to the magnitude of the failure, and is also dependent on the number of vehicles using the road section. Another important factor to consider is the configuration where the failure is occurring. A loss of skid resistance is likely to be dangerous near an intersection, whereas deformation is likely to be most dangerous on a highway where the speed limit is 100 km/h.

In addition, alignment and cross sectional details such as curve, grade and road and shoulder widths would all influence the risk to road users. Accident data may be checked to see the accident history in the section, but often no definitive conclusions can be made, due to the often short time in which a failure occurs.

Risk of further deterioration in the future

In the early stages of an investigation, the risk of further deterioration of a pavement failure is often difficult to determine. Generally, however, if a failure has occurred fairly rapidly, there is a good chance that further deterioration will also occur fairly rapidly.

Conversely, a road that has failed over a large time, such as 10 years, is unlikely to have any large rate of deterioration in the future. However, with all pavement failures, the rate of failure generally increases as the failure gets worse, and this should be taken into account.

Planned Future work

There is little point in carrying out a failure investigation and minor rehabilitation work, if a major piece of rehabilitation work is planned for the near future. In this case, an investigation may provide valuable information as a guide to the rehabilitation work, and often the road may be able to 'hang on' until the major work is completed, this being dependent on the factors listed above.

3.3.4 Investigation Plan

Planning the Investigation is an ongoing process through the investigation, and may be modified as required. Important steps are discussed.

Goal Setting

When carrying out an investigation of a pavement failure, it is important that there is understanding of the required goal of the work. For example, is the failure being investigated for increased understanding to prevent reoccurrence in the future, or is the primary focus on the selection of the best rehabilitation treatment.

The goal is often related to the scope of work required, and so this should be taken into account when setting goals. In addition, the goal may need to be modified as the investigation progresses, due to extra information available.

Budget

Once the goal and scope of the work has been roughly determined, a budget can be drawn up. Costs to be included might include testing costs, personnel costs (if specialised personnel such as traffic control must be hired) and any other costs that might be relevant.

It should be noted that often the cost of testing and laboratory usage is minor, compared to other costs in the investigation. In addition, consideration should be given as to preliminary costing of possible rehabilitation work.

Operations Planning

Operations Planning refers to integrating and coordinating the activities of all investigation team members. This is best done using periodic meetings to review findings and discuss courses of action, and good communication is essential, especially if as is often the case, many of the team members are also working on other tasks simultaneously, and cannot devote their full attention to the investigation.

Investigative Synthesis

In the latter stages of the investigation, when much investigation and testing has been done, and the probable cause of failure is being considered, all the information that has been obtained must be linked together in a logical and coherent way, to ensure no important details have been missed.

The simplest way to do this is to prepare a report listing details about the project, inspection and testing carried out and their results. All information obtained should be listed, and feedback from other team members will ensure nothing important is missed.

3.3.5 Investigation Team

The investigation team composition may vary widely, depending on who is available and the current workload. However, personnel selected should be knowledgeable in the area. It is also important that the team leader is identified at an early stage. This person should be responsible for the overall management of the team to ensure the goals are achieved. Testing and maintenance personnel are not considered here, but also should be trained in their task.

The investigation team members may consist of a variety of personnel, as depending on availability and circumstance. Maintenance engineers can provide information and knowledge about various rehabilitation treatments, and well as information about similar failures in the past.

If the failure was a premature failure on a recently completed project, the construction engineers and inspectors are especially important, due to their knowledge of the project and conditions. The engineer may know important administrative and procedural details, while the inspector may know about the day-to-day work on the project

In the case of a failure not directly related to a specific project, the local area engineers and inspectors would provide a similar function and information to that listed above for the construction engineers and inspectors, due to their knowledge of the local area.

The laboratory or materials officer may provide two functions. Firstly, to organise and coordinate any testing required, and secondly, to provide their knowledge about the behaviour and properties of materials.

In addition, various other personnel may be included in the investigation, as circumstances require. For example, in the cause of a major failure in Queensland, specialised personnel from RS&E (Road Systems and Engineering) may provide much of the knowledge and resources for the investigation.

3.4 Review Documents and Literature

3.4.1 Introduction

This section discusses documents and literature that should be reviewed when investigating a pavement failure. The information found in this step may often give a good indication of which testing should be carried out, or even negate the need for any testing at all, if a probable cause of failure is found at this stage.

3.4.2 Plans

A major road would typically have plans available for at least the most recent work. In addition, some information about layers and thicknesses may be available off an asset management program. For example in the Queensland Department of Main Roads, information is available from the ARMIS system, using the Chartview program.

Important information to gather would include typical pavement cross-sections, types of materials used, layers and thicknesses and crossfalls. Information about the surfacing, such as binder and aggregate application rates may also be important.

Other information to get off plans may include location of services (if any), design life of the road, design traffic loading, and design material strength values adopted. In addition, curve and grade information may be required, as well as information about cut and fill locations. This information may enable a rough check of the pavement design to be done (when traffic data is available) to ensure a sufficient pavement thickness was used.

3.4.3 Pavement History

This information may often be obtained from an asset management program, such the ARMIS system, used by the Queensland Department of Main Roads.

Important information might include the date of initial construction, and any work carried out since, as well as the seal and pavement age, and the depth and types of materials used. Additional information that might be useful is data related to failure magnitude, such as roughness and rutting, and maintenance costs in the past.

The information may be observed visually from the program interface, but for a more detailed analysis, it is possible to import the data into a spreadsheet.

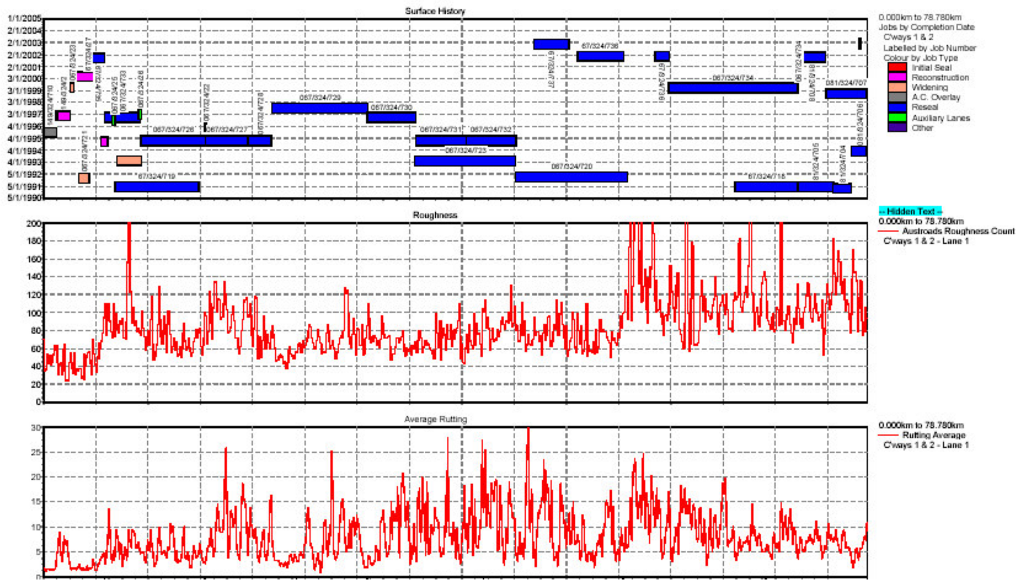


Figure 3.1 – A view of typical information available from an asset management program, showing surface history, roughness and average rutting along the length of the road

3.4.4 Drainage Design

This information is generally available off the plans, and should be noted, for comparison with on site. Drainage is important for any road, and the failure or inadequacy of the drainage system allowing excessive moisture into the pavement is often the cause of a pavement failure.

Features to take note of would include details about culverts, table and catch drains, and any subsoil drainage. The in-service function of the components of the drainage system must be verified on site.

In addition, consideration should be given to the factors that influence the moisture condition of a road, and these factors can be further investigated on site.

3.4.5 Pavement Materials Information

Type of Material(s)

Information about the material type(s) is often available off the plans. Some information about application rates (e.g. bitumen) may also be available from here, and should be compared to that used on site. Knowing material types may give an indication of possible failure causes. For example ridge gravel may be susceptible to crushing, whereas this seldom occurs in crushed rock gravel.

Required specification of material properties

Specifications for material properties are given by most road agencies. For example, the most important Queensland Department of Main Roads Standard Specifications are as follows:

- MRS 11.04 - General Earthworks
- MRS 11.05 - Unbound Pavements
- MRS 11.17 - Bitumen
- MRS 11.18 - Polymer Modified Binder
- MRS 11.22 - Supply of Cover Aggregate
- MRS 11.30, 11.34, 11.36 - Asphalt Pavements (varying types)

Other specifications or standards may need to be used in varying situations, as required. For example, if a road agency does not have a specification required for a particular area, information may need to be sought from other sources.

It should be noted that compliance with the specifications does not guarantee that a pavement will not fail. Instead, the specifications are continuously changing to reflect increased knowledge of pavement materials, and are mostly a guide to what should provide a satisfactory pavement.

In recent years, there has been a gradual shift in materials testing from method-based specifications, such as the use of empirical tests, over to performance-based specifications, where material properties are tested, and specifications are based on performance, not the method used.

In addition to the specifications, other conditions on material properties may be listed in the Supplementary Conditions of Contract. These conditions may have been designed to allow the pavement to cope better with local conditions.

Material test results

These may be available from varying sources depending on who carried out the testing. Test results may be on file, and the local Laboratory or Materials Officer may also know of test results. This might include test results from during design, construction, or afterwards.

History of use of similar material

This knowledge may be available from the Laboratory or Materials Officer and other knowledgeable people such as the Maintenance Engineer or Inspectors may also know of details. It may be important to know of any previous failures in this material type.

3.4.6 Construction records

Testing methods and frequencies

Required testing methods and frequencies are often specified in the contract conditions, and a check of the testing carried out may be made to see if the test methods and frequencies are satisfactory. If they are not, this may have meant that there was insufficient control over material variability in the project.

Other Information

It is important to note the rate of application of the various road-related materials used in the project, including binder and aggregate, and any cutter, precoating or adhesion agents used.

The project inspector would normally be able to provide access to any construction diaries, or visual descriptions of the materials. This information and may provide an indication of possible problems during construction, or with the materials used.

Important dates and correspondence during construction would normally be available from the Project Inspector or Engineer, and may highlight problems during construction.

Finally, any changes of the initial design for construction reasons should be carefully noted, since they may have had unforseen consequences, reducing the performance of the pavement so that failure could occur.

3.4.7 Traffic Data

Information regarding traffic volumes is important information, and in most cases the AADT and % of Heavy Vehicles data is sufficient. In the Queensland Department of Main Roads, this information is available from Traffic Census data. Important data to note is:

- AADT (Average Annual Daily Traffic)
- Traffic Composition, especially Heavy Vehicles

The traffic data may provide a guide as to whether the pavement failure is likely to be related to traffic volumes or other causes. For example, on a road with AADT=10,000 vehicles including 1500 heavy vehicles, traffic loading on the pavement is likely to be a major contributing factor to the pavement failure.

Conversely, a road with only 20 vehicles/day including 10 trucks is unlikely to fail solely due to the traffic loading, and environmental conditions may be an important factor. However, this will depend on the pavement design.

3.4.8 Soil or Geological Records

Soil or geological records may prove useful in a limited number of cases. However, due to the variation in soil and geology, this information is normally best obtained on site.

3.4.9 Temperature, Weather and Rainfall Data

Temperature data may be of limited importance, if it is thought that temperature is a factor. Measurement of the actual road temperature is more important than air temperature, due to the difference between these values. Important data that may need to be collected includes the temperature of the road at the time of sealing and the general daily temperature range at this time.

Seasonal Moisture patterns may be dependent on the land use adjacent to the road and weather. It may play a part in the pavement failure. Information about heavy rainfall during construction and remedial measures (if any) to mitigate its effect are normally available from construction diaries (available from Project Inspector).

Information about heavy rainfall since construction may be available from the Area Inspector, or possibly the Bureau of Meteorology, although rainfall is so variable that this may be of limited value.

3.4.10 Published Articles

Road Agency

The road agency carrying out the failure investigation may have published articles that are of relevance. In addition, publications may be available internally within the organisation.

For example, various articles are published by the Queensland Department of Main Roads, and may be available from the website. Additional articles are also available from specialist areas in RS&E (Road System and Engineering).

These articles may provide useful information about the failure and its possible causes, as well as similar cases in the past.

Other Sources

The company Austroads publishes information about roads and pavement issues in Australia, and some articles are available from their website. These may provide information that could be useful, although consideration must be made of whether it is relevant to local conditions.

Other road agencies, both overseas and in Australia, may have information available that can be of use. These may be available from the websites, or other sources.

Other sources of potentially useful information included published journal articles or articles available from the Internet. Providing account is taken of the varying conditions and testing used in different locations, these sources can be a valuable reference tool.

3.5 Interview Personnel

3.5.1 Introduction

This section discusses personnel that should be interviewed when investigating a pavement failure. The primary reason for doing this is to benefit from the knowledge and experience of others. This will help to ensure that the cause of failure can be reliably found, and the best rehabilitation option selected.

Often, these personnel may have knowledge that while not significant enough to be documented, may provide an indication of the possible cause(s) of the failure.

3.5.2 Designers

Designers are useful sources of information about the design of the project. Important details to get from them are details about any studies made of local conditions, and any assumptions made during the design. Incorrect assumptions, due to a lack of knowledge about the site conditions, may have caused or contributed to the failure.

In addition, the designer should be asked about what they think is causing the failure, since they may have some insight into the design aspect that is relevant.

3.5.3 Construction Personnel

Important personnel to interview might include the project engineer and inspector, and the laboratory or materials officer. These personnel may be able to provide information such as rates of application of various materials, testing methods and frequencies, a visual description of the materials used, and access to construction diaries.

In addition to the above information, they should know of important dates and events during construction, any important correspondence, and any changes made to the pavement or surfacing design for construction reasons.

3.5.4 Maintenance Personnel

The district maintenance engineer may have valuable knowledge about similar failures in the past and their causes, as well as the best rehabilitation treatments. Maintenance workers may be able to provide firsthand information that may be important.

3.5.5 Other Personnel

Various other personnel may have information about the failure that should be looked at. In all cases, the accuracy and validity of the information should be carefully examined.

3.6 Non-destructive Condition Survey

3.6.1 Introduction

This section discusses non-destructive methods for assessing the condition of a pavement, and examining the pavement failure that is occurring. This may help to determine the need for rehabilitation work, and provide an indication of the probable cause of the pavement failure. It will also provide information about possible factors that may influence the rehabilitation option selected.

Non-destructive is taken to mean testing and investigation methods that do not require the removal, disturbance or destruction of any part of the pavement structure.

3.6.2 Visual Examination

In many pavement failures (especially minor ones) the only form of investigation of the actual failed pavement is a visual examination. This is due to the minimal cost of doing this, compared to other more expensive forms of deflection testing and materials sampling and testing. For this reason, an effective systematic method of visual investigation of pavements is essential, to ensure that the cause of the failure can be diagnosed efficiently.

In pavement failures where some form of testing will still be carried out (more major failures), visual examination is still vitally important, since it is a guide as to what testing should be carried out and where. In addition, it will provide valuable site-specific information that may have an influence on the best rehabilitation treatment.

Due to the cost of testing, it must be selected to ensure that the most useful information is obtained at the least cost, and a comprehensive and detailed visual investigation of the pavement will help to ensure that this is achieved.

Ability to carry traffic safely

In the case of an extreme pavement failure, the road may need to be closed, or the speed limit reduced. This need may be assessed by checking the roughness of the road, or any other safety hazards such as loss of skid resistance or large deformation

Examination of the Pavement Failure

A detailed record of the pavement distress is useful for later reference so that the failure and its details can be remembered off site. In addition, people who do not visit the site are able to get an appreciation of the failure.

The best way to record the details of the pavement distress is a written description, sketches of the failure and photos. In addition, simple measurements using a straight edge or level will be useful, if there has been a change in geometry. Perhaps, if more detail is required, a specially equipped asset management/condition survey vehicle could be used for defect mapping.

Important information that should be noted when examining a pavement failure will include the following:

- Is the failure constant throughout zone or only isolated?
- What is the extent in both directions, and is it occurring on both sides of road?
- Where is the location of the failure on the profile of the road?
- Are there any other types of pavement distress that may be related?

It is important to check if the failure occurs continuously throughout the failure zone or only in isolated sections. This can give valuable information regarding the cause of the failure, and whether it is systematic or only sporadic.

For example, constant failure throughout means it is probably a systematic failure due to a dominant condition (which may still have many causes), whereas failure in isolated sections could be due to random or systematic conditions that cause the failed sections of road to be different to the others in some way.

Notes should be made of the length of the failure, and whether it occurs on both sides of the road, or only on one side. This may give information regarding the failure cause.

The location of the failure on the road profile should be noted i.e. is it occurring in wheelpaths, and if so, which ones? This may give a clue as to whether the cause of failure is likely to be traffic related, environmentally related or a combination. A careful eye out should be kept for other types of pavement distress that may be related to the failure, as this may indicate a probable cause of the failure.

3.6.3 Drainage and Moisture

Poor drainage is often the cause of pavement failure, since excess moisture leads to a reduction in the strength and stiffness of the materials, and the possibility of pressure-induced erosion.

For this reason, the drainage of the road should be carefully assessed, and this is best seen during or soon after a rain event. The *Pavement Rehabilitation Manual* (Queensland Transport, 1992) provides a good introduction to this area.

Factors influencing moisture condition in a Pavement

The factors affecting the general drainage condition of the road include the position of catch, table and subsoil drains, shoulder crossfall, longitudinal grade, type of shoulder (vegetated, bare, sealed), the formation profile and whether the road is constructed on cut or fill.

The water table position may be static or variable, and could be deep or shallow. Often a shallow variable watertable may cause problems, due to the moisture changes that this can cause in the road pavement.

Climate factors such as rainfall, evaporation, temperature and thermal gradients may influence the moisture condition of the road. For example, large rainfall may mean that drainage systems cannot cope, so water is unable to drain out of the pavement.

The type of construction methods used influence the moisture condition of the pavement. Important influencing factors include whether the pavement is boxed/trenched or untrenched, and the moisture contents and degree of saturations of the various materials at compaction or sealing.

The surrounding landform can influence the moisture condition of the road pavement, depending on how much moisture migrates from these areas to near the road. Important landform details to take note of might include drainage depressions or swamps, adjacent rivers or irrigation areas, extent of vegetation and type, and the runoff and permeability of soil strata.

The type of materials used in the pavement can influence the effects of moisture on the performance of the road. Important properties might include grain and pore size, density, mineralogy, shrink-swell properties, permeability and the extent of salinity.

Moisture entry locations

Sources of water entry into the pavement should be assessed, since water entry may be the primary cause of failure, or a major contributing factor. The main sources are as discussed below.

Moisture entry through the road surface may occur due to unsealed shoulders, inadequate pavement surface drainage during construction, exposure of surface to rain during construction, porous hot mixed asphalt, or pavements left primed but not sealed for extended periods.

Moisture entry through the side may occur due to inadequate drainage allowing pondage of water that migrates into the road pavement.

Construction practices may contribute to excessive moisture in the pavement. This may be caused by the use of a high moisture content for compaction, or excessive watering of the surface (due to surface finishing requirements and/or backwatering to maintain the surface).

Moisture entry into pavement may also occur due to other causes, such as seepage from groundwater, movement of a water table under a road, or capillary water movement.

Effectiveness of drainage in controlling moisture

To evaluate the effectiveness of the road drainage system in controlling moisture, it is necessary to note any shortcomings that are occurring, and possibly contributing to degradation of the road pavement. The most important shortcomings that may occur, allowing moisture entry into the pavement, are discussed below.

Problems on surface of road

If the surfacing is permeable, allowing moisture to infiltrate, then water may be trapped within the pavement. This may occur especially if there is no drainage within the pavement structure (e.g. subsoil drainage). This will lead to a loss of strength in the pavement.

Sheet flow over the surface of the road may occur during times of heavy rainfall. This is excessive water on the surface leading to spray and splash, and possibly aquaplaning. This could be due to the pavement not allowing water to drain off quickly enough, or the ineffectiveness of drainage structures in preventing water reaching the road surface.

Water ponding may occur on the road surface due to incorrect surface crossfalls, or changes in geometry, such as depressions. Cracks, joints, potholes and other discontinuities in the road surface may allow moisture infiltration into the pavement.

At sag vertical curves where two down slopes meet, moisture may accumulate on the surface of the pavement, especially if it cannot drain sideways off or away from the road. Moisture may also accumulate at changes of pavement type, thickness or patches occurring on longitudinal grades.

Problems at side of road

Shallow and/or silting table drains may not allow moisture to drain through them, and may produce softening of the shoulders and subgrade. Permeable shoulders and medians can allow moisture entry into the pavement, especially if water is allowed to pond.

Water seepage into pavement may occur adjacent to the median. For example, the high permeability sand fill under concrete slabs may act as sponge feeding water into pavement. Moisture infiltration from a cutting may occur, especially if the soil and rock are of varying permeabilities leading to problems with sub soil water.

Problems due to construction practices

During the construction of a road widening, subsurface drainage outlet pipes may be accidentally blocked. In addition, if low permeability base material is used in the widening, it may inhibit water drainage from the existing base.

The drainage capacity of kerbed pavements may be reduced, due to overlays, since the overlay reduces the area available in the kerb gutter for water flow. Impermeable shoulders (boxed construction) mean that any water that enters the pavement may not be able to escape.

Problems due to other sources

Impermeable aggregate drainage layers may allow prolonged wetting of subbase and subgrade. This may cause the movement of fines into the subbase and base.

Figure 3.2 – Problems with the drainage system:

a) A culvert blocked by vegetation

b) A culvert that no longer functions effectively

Pockets of unstable subgrade may be the cause of localised pavement distress patterns. Peat pockets, organic or deleterious matter, localised springs, groundwater seepage or inoperative subsurface drainage systems may cause these isolated pockets. Broken and clogged pipes and pipe outlets retard or inhibit flow through drainage system. The affected region may be localised.

Salinity and Rising Water Tables

This will prove an increasing problem in Australia in the future. For detailed information about this, see *Salinity and Rising Water Tables — Risks for Road Assets* (Austroads, 2004).

3.6.4 Topography and Alignment

The topography should be checked to see whether it is flat, rolling or mountainous. The alignment should be checked to see whether it is straight, gently curved or windy. The road geometry should be examined, including a check on whether the road is constructed on cut or fill.

Steep grades may cause greater forces to be exerted on the road e.g. heavy braking. The type of alignment may have an influence on where the vehicle travel paths are, and the forces on the pavement from vehicles may also vary.

The crossfalls of the lanes and shoulders should be measured and compared with the plans. Any deviation may be due to a reactive subgrade, and change in crossfall could be contributing to the failure. In addition, any other deviations from the plans should be noted.

Failures may vary slightly, depending on whether material is on cut or fill, and possible moisture entry sources are slightly different.

3.6.5 Soil and Geology

Soil types can be roughly determined using simple and quick manual classification tests. This may indicate possible sources of problems, such as highly expansive clay materials. In addition, it should be noted whether the soil is moist or dry, as excessive moisture may be a sign of poor drainage.

The following is a simple means of classifying a soil, adapted from the *Road Drainage Design Manual* (Queensland Department of Main Roads, 2002b). Other methods may also be used, as preferred.

In a visual examination however, the main purpose is just to get an indication of possible behaviour and drainage of the soil, to determine what may be the most suitable testing. If more detailed information was required, samples could be taken and analysed.

Soil Texture

This is easily measured by creating a soil ball using water and trying to create a ribbon from this (extruding a ribbon of soil under pressure from between a clenched forefinger and rolling thumb). It can provide a quick and rough indication of the main types of soil particles present in the soil.

Major texture groups and descriptions are listed below. Soil profiles may vary with depth in which the soil may be classed as gradational; if the profile is constant over the depth, it may be classed as uniform.

Sands (sand, loamy sand, clayey sand) have nil to slight coherence, with a sandy feel. A ribbon is difficult to form, with a normal length <15mm. Sandy loams form a coherent ball that is very sandy to touch. The ribbon may have a length of 15-25mm.

Loams (loams, silty loams) form a coherent and spongy ball, which has a smooth to silky feel with no sandiness. The ribbon may have a length of about 25mm. Sandy clays form a strongly coherent ball, which is sandy to touch. A ribbon of about 25 - 40 mm length may be formed.

Clay Loams (clay loams, sandy clay loams, silty clay loams) form a coherent ball, with a smooth to silky feel. A ribbon of about 40-50 mm long should be able to be formed. Light clays (light clays, light medium clays) form a plastic ball that is smooth to the touch. There is a slight to moderate resistance to ribboning, but a length of 50 - 75 mm should be able to be formed.

Medium-Heavy Clays (medium clays, medium heavy clays, and heavy clays) form a smooth plastic ball, like plasticine. There should be a moderate to firm resistance to ribboning. A length of >75 mm should be able to be formed.

Soil Colour

Soil colour may provide an indication of the drainage status and permeability of a soil. Red soil is generally well drained, and may indicate high – moderate permeability.

Blotchy or continuous white subsurface soil is indicative of restricted permeability. It also may indicate a seasonal perched watertable on top of clay subsoil. Dark grey or olive grey soil may indicate slow permeability

Soils that are saturated for long periods are often dominated by olive grey, bluish grey and greenish grey colours, although patches of orange or red mottles may also occur.

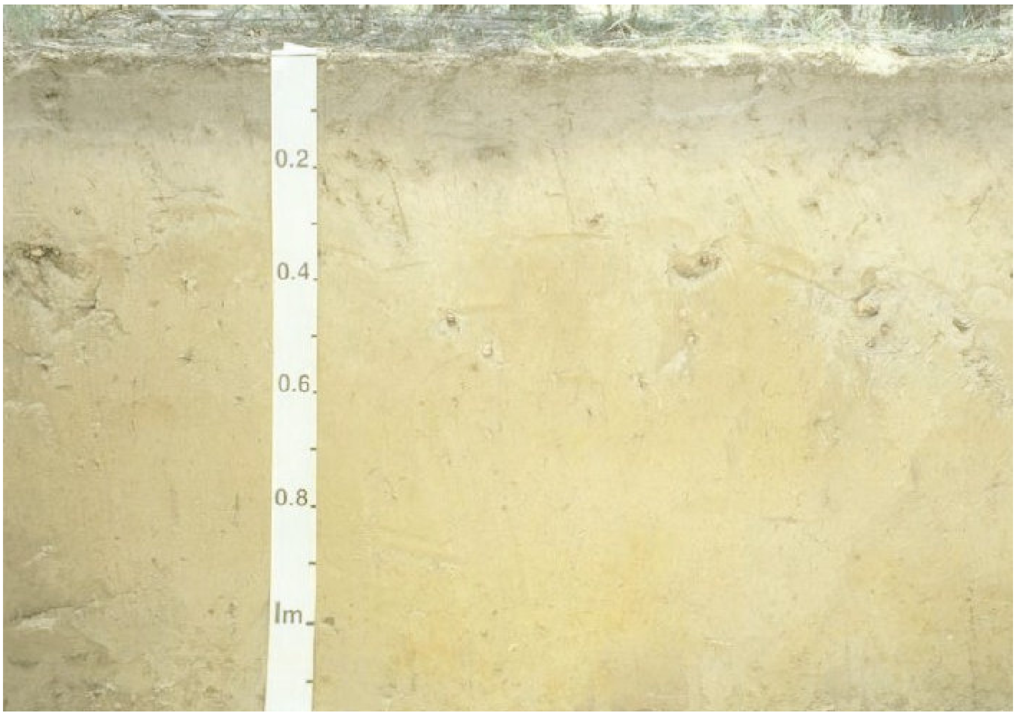


Figure 3.3 – Typical soil profiles:

a) A uniform sand or sandy loam soil

b) A sandy gradational soil

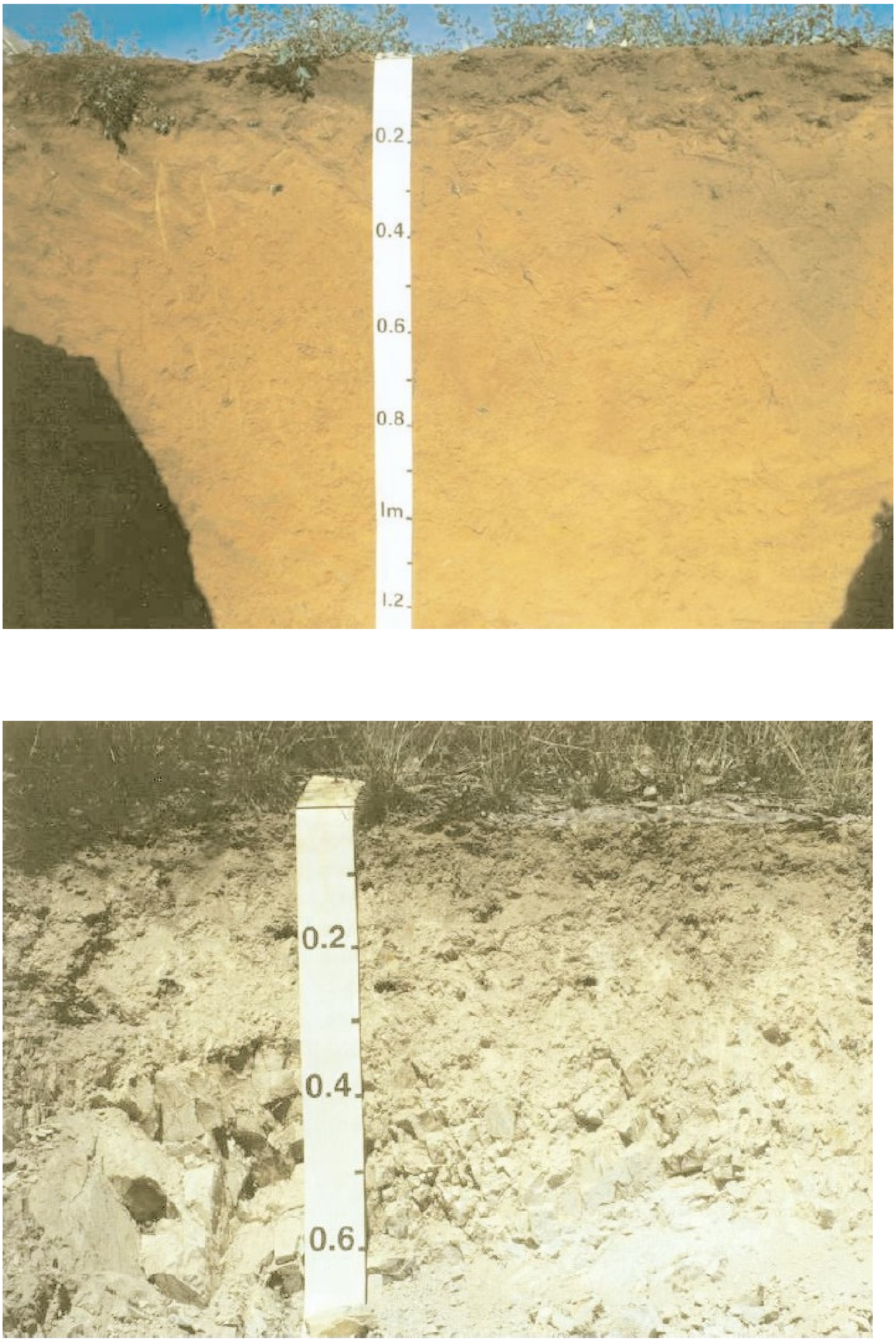
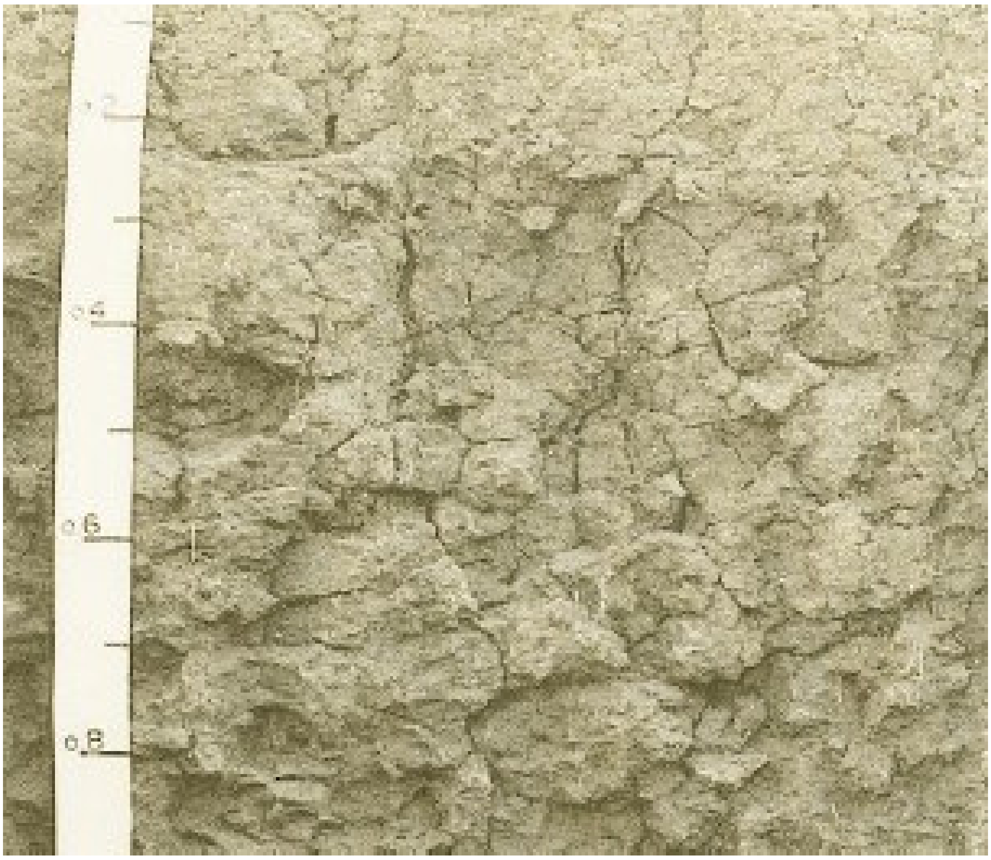


Figure 3.4 – Typical soil profiles:

a) A loamy gradational soil

b) A uniform loam or clay loam



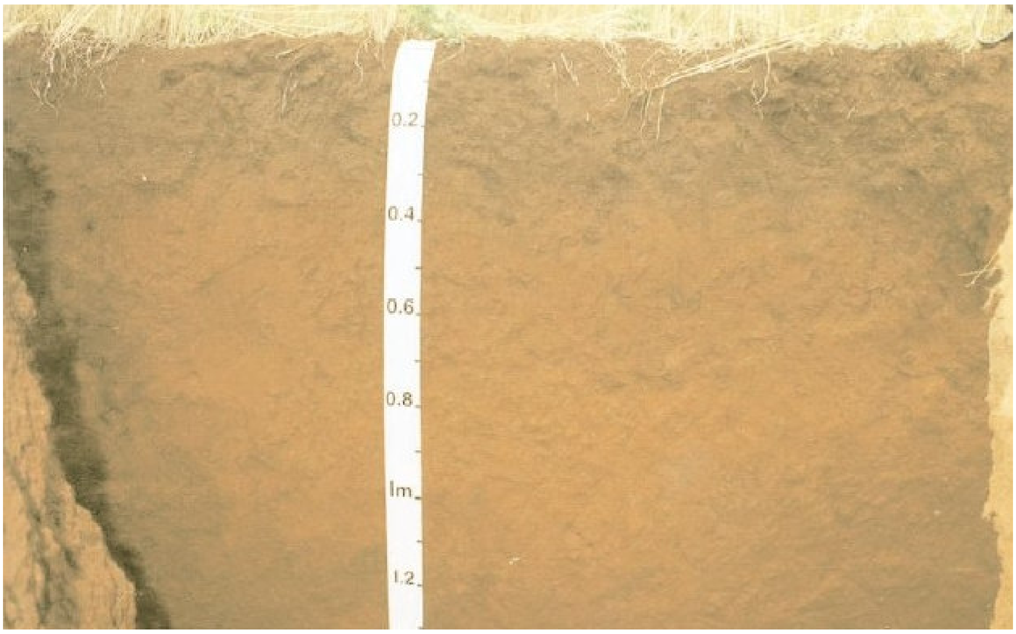


Figure 3.5 – Typical soil profiles:

a) A uniform cracking clay

b) A uniform non-cracking clay

Geology

In some cases, the geology of the rock near the road may be important. In this case, studies should be made as required. Generally, any contribution to pavement failures is minimal, since rock is often much stronger and stable than soil.

3.6.6 Deflection Testing

Introduction

Deflection Testing can assess the strength of the pavement by indirect means i.e. how the pavement structure deflects when a loading is applied. The advantages of this form of testing include that the pavement structure is not destroyed during testing, negating the need for work to fix any damage to the pavement caused by testing.

The disadvantages of this form of testing include the cost associated with its use, the obstruction to traffic, and the need for specialised personnel and equipment. There is also some uncertainty about interpretation of the results.

When using deflection testing, there may be a need to convert deflections using one method to the equivalent deflections using another method. Guidance on this matter is provided in *Pavement Strength in Network Analysis of Sealed Granular Roads: Basis for Austroads Guidelines* (Austroads, 2003a).

Providing its use is warranted, deflection testing can provide valuable information about zones of weakness within the structure of the pavement. The main deflection testing machines used are as follows:

Network Level Deflection Testing Machines

These are deflection-testing machines that are used for large network surveys to find locations requiring more detailed testing of the pavement. Hence, they would probably not often be used for detailed examination of a small failed pavement section, although for sections longer than about 1 km they would be useful.

Different names and devices may be used, including Deflectograph (in some of Australia) and PAVDEF (in Queensland). A general configuration of such a machine is similar to a pair of small Benkelman beams (see below), mounted on sled behind rigid truck. Testing may occur at a speed of about 3-4 km/h.

Benkelman Beams

These devices were widely used in Australia, although their use is becoming less common. They are labour intensive, but produce detailed data, allowing for more critical analysis. Their use is generally only recommended for pavements up to 450 mm thick and testing is only suitable for small areas, due to the slow testing speed. A moving load is applied to the road surface, and the deflection at various points is measured.

Falling Weight Deflectometer (FWD)

This is the most accurate and costly of the deflection testing machines commonly available. It is limited in its ability to test top 200 mm of the pavement, so it is best suited to deep pavements, where the top 200 mm is uniform. As suggested by its name, an impact load, rather than a moving wheel load, is applied to the pavement. Heavier weights can be engaged for stiffer layer analysis, such as on bound pavements. The FWD is widely used in the USA also.

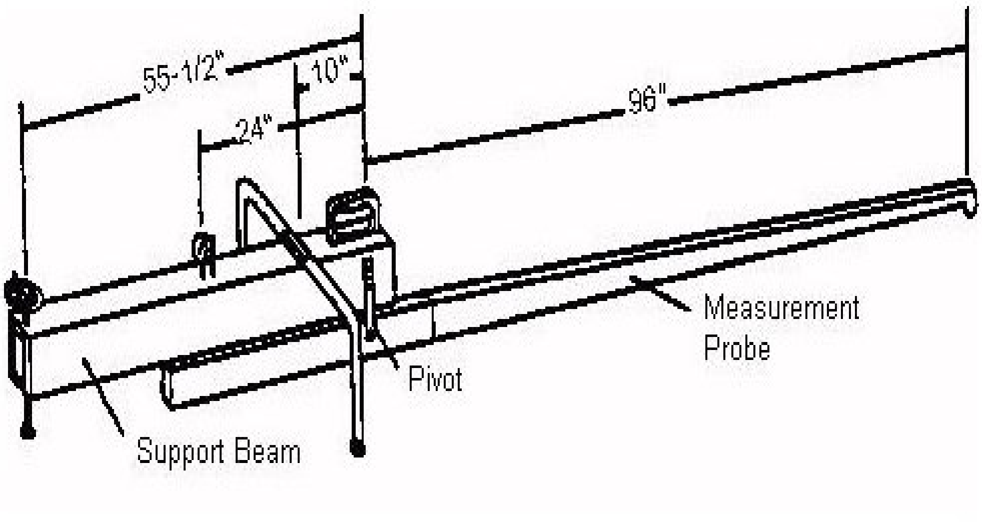


Figure 3.6 – Benkelman Beam:

a) A schematic of a typical apparatus

b) In use

Figure 3.7 – The Falling Weight Deflectometer (FWD):

a) A view of the complete trailer

b) A view of the impulse loading mechanism (at the left) and sensors (at the right)

Interpretation of Deflection Testing Information

The interpretation of deflection testing information is best left to people knowledgeable in this area, due to the possibility of incorrect interpretation of the results.

However, a short introduction to this area will be given below, adapted from the *Pavement Rehabilitation Manual* (Queensland Transport, 1992). This information was written for Benkelman Beam test results, and as such, some of the information may not be relevant for other testing devices.

A typical view of the deflection occurring in a pavement due to a passing wheel load is shown in the figure below. The parameters that are most important are shown in the table.

It is suggested that testing should be carried out in both outer and inner wheelpaths, and staggered between adjacent lanes. The maximum recommended testing interval is 50m for detailed testing, although for network level surveys, the interval may be 100m or larger. Corrections to results for temperature, speed and moisture condition should be made as required.

Table 3.6 – Important parameters in deflection testing

Parameter	**Details**
Rebound Deflection	D_0 = Maximum deflection – Residual deflection
Deflection Ratio	$D_R = D_{250} / D_0$ (may also be a percentage)
Curvature Function	$C_F = D_0 - D_{200}$

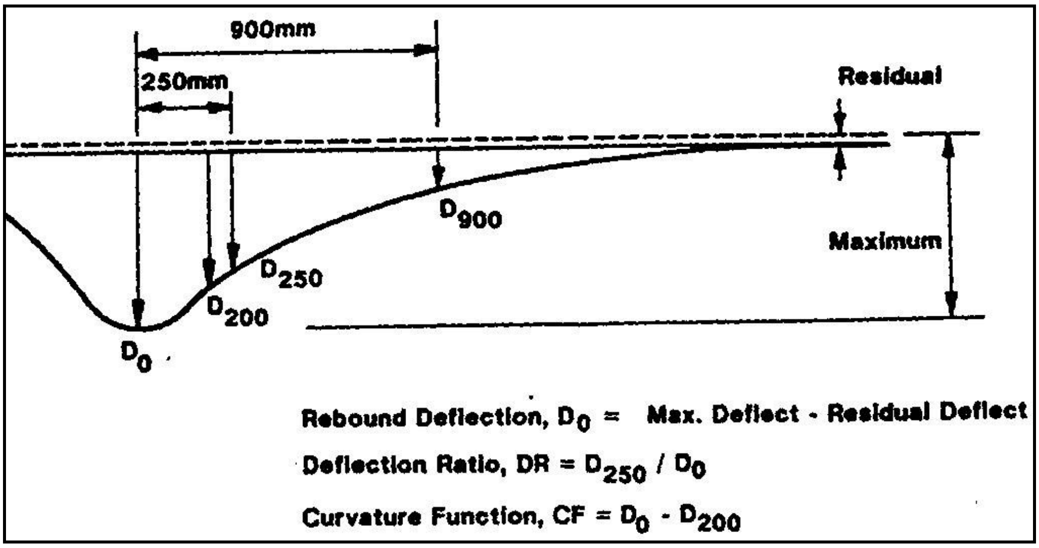


Figure 3.8 – A typical deflection bowl in a pavement subjected to a loading

The pavement should be split into separate homogeneous subsections, based on deflection testing results. Homogenous is taken to mean the coefficient of variation (% standard deviation/mean) is less than 30%. The following terminology is used:

X Mean of deflection parameter for a homogenous section
SD Standard deviation of deflection parameter for a homogenous section
CV Coefficient of variation = SD / X x 100

Representative Deflections (D_r)

The Representative Deflection (D_r) for a homogenous section corresponds to the 90% highest Rebound Deflection (D_0), and is calculated for both inner and outer wheelpaths. The representative deflection for design is the larger of these two values, calculated as

$D_{r\,(I\text{ or }O)}$ = X + 1.28 SD

Residual Deflections

This is the deformation of test site after test vehicle moves beyond range of influence, the permanent deflection. It can generally only be measured using a Benkelman beam apparatus. Readings of either >0.15 mm or >25% of maximum deflection may indicate weakness in upper pavement layers or deformation of surfacing (this is not the case if the pavement is rigid or bound, such as if cement treated).

Deflection Ratio

The representative Deflection Ratio for a homogenous section is the 10% lowest value of the deflection ratios (which may be expressed as either a decimal number or a percentage):

$$D_R = X - 1.28\ SD$$

The deflection ratio may be used to estimate the structural integrity of the pavement, although this is difficult to do due to the complexity of behaviour of different pavement types with varying thicknesses.

For Benkelman Beams, the *Pavement Rehabilitation Manual* (Queensland Transport, 1992) recommends the following about the interpretation of deflection ratio values, shown in the table.

Table 3.7 – Pavement Rehabilitation Manual: Interpretation of deflection ratio

Deflection Ratio	Interpretation
0.8	A rigid or bound pavement
0.6 – 0.7	Expected for a good unbound pavement
< 0.6	A possible weakness in the pavement

These values were based on work done by the Australian Road Research Board (ARRB) in 1979, but the results appear to have been misinterpreted. From an examination of the original work (Scala, 1979), the following was deduced, regarding the deflection ratio parameter, shown in the table.

Table 3.8 – Interpretation of deflection ratio based on original work

Deflection Ratio	Information
0.6	Average for 500 sites: Victoria and New South Wales
0.4 – 0.65	Range for all pavements
0.55 – 0.65	Range for most pavements with depth > 150mm

For the PAVDEF (Deflectograph) deflection testing device, Baran (1994) suggests that the following criteria may be considered, based on the analytical analysis of typical pavement configurations and supported by limited field correlation:

Table 3.9 – Interpretation of deflection ratio for PAVDEF (Deflectograph)

Deflection Ratio	Interpretation
>0.6	Representative of a bound base
0.4 – 0.5	Unbound Granular Base
< 0.4	Could represent a weakness in the base layer

From the ARMIS database, the 1999 PAVDEF deflection testing results for the Warrego Highway (18A) between Ipswich and Toowoomba were examined, to determine what values for a predominantly unbound granular pavement may be usual. The test results were averaged over 100m intervals. There was a length of 95 km, and about 3390 values (from varying carriageways, lanes and wheelpaths). The results are as shown in the following table:

Table 3.10 – Deflection ratio for PAVDEF along the Warrego Highway: Ipswich – Toowoomba (18A)

Deflection Ratio	Information
0.30	15% Percentile Value
0.39	Average and Median Value
0.47	85% Percentile Value

Curvature Function

The Representative Curvature Function for a homogeneous pavement section is the mean of the test curvature functions. In general, high values (>0.4mm) may indicate a lack of stiffness, a thin pavement or cracked surface. Low Values (<0.2mm) indicate a stiff pavement.

Subgrade Response

For all but bound pavements, the Benkelman Beam D_{900} value reflects subgrade response unaffected by overlying pavement, and can be used to estimate subgrade CBR. The representative D_{900} value for a homogenous section is the 90% highest $D_{900:}$

$$D_{900} \quad = \quad X + 1.28\ SD$$

Table 3.11 – Correlation between D_{900} value for Benkelman Beam and subgrade CBR

D_{900} (mm)	0.33	0.23	0.18	0.145	0.11	0.09
CBR	3	5	7	10	15	20

For other types of deflection testing device, it is possible to establish a correlation that enables the subgrade CBR to be estimated.

Back Calculation of Layer Moduli

With forms of deflection testing where the deflection bowl is measured at a number of points, it is possible to back calculate the layer moduli. To do this, it is typical to use a computer program, ands it is necessary to assume information about the layers, such as the thickness and design moduli.

This calculation can give some information about any possible weakness in the pavement or the subgrade, although it should be noted that there is often more than one combination of layer moduli that satisfies the required deflection bowl shape, and so the results obtained can be open to some interpretation.

3.6.7 Other Forms of Non-Destructive Testing

This section describes other forms of non-destructive testing that may be used to assess the condition of the pavement. Various testing methods are used in different places, so only a general introduction to each test is given.

Seismic Surveys

These tests use a seismograph or similar device to measure seismic vibrations, providing information about the pavement structure. It would be of limited use, except for specialised applications.

Roughness and Surface Evenness

These tests measure the roughness and surface evenness of the road. This may be useful for examining the roughness levels of the pavement, since the magnitude of this may influence the level of action required in rehabilitation work, and the urgency required.

Roughness and surface unevenness may be measured using various devices depending on circumstance. These include laser profilers, roughness meters, manual roughness measuring devices, and digital or mechanical profile beams.

Figure 3.9 – A Network Survey Vehicle used to assess pavement properties such as roughness and rutting at the network level

Figure 3.10 – A portable skid resistance (pendulum) tester, used for assessing skid resistance of a road surface, usually only over a small area

Pavement Lateral Profiles

These tests measure the lateral profile of the pavement. It may be useful for examining the size of rutting occurring. Various devices may be used, including 'rutmeters', profile beams or simply a straightedge of some length.

Skid Resistance

These tests measure the skid resistance of the pavement. It may be useful for examining the skid resistance levels of the pavement at specific locations, especially those where a loss of skid resistance is likely to be catastrophic, such as near an intersection.

Various devices are used, depending on circumstance and the size of the area required to be tested. These include portable pendulums, locked wheel testers, fixed slip testers, and the Sideways Coefficient Routine Investigation Machine (SCRIM), which is best suited for large areas.

Ground Penetrating Radar

This testing method is widely used in the USA (Chen et al., 2003, Mooney et al., 2000, and Victorine et al. 1997). It is also being introduced in Australia and Queensland, with trials being conducted to assess its suitability for use in the testing of timber, concrete and pavements (Muller, 2001, and Muller et al., 2001).

Experienced personnel are required to operate the apparatus and interpret the results obtained. For pavements, information can be found regarding defects in layers, changes in construction, layer thicknesses, subgrade features, approximate moisture contents, or evidence of moisture damage in layers

X-Ray Diffraction

This test uses X-Rays to identify chemicals present in the material. It may be used in some cases to identify chemicals present in the material or to identify whether certain clay types are present. This may provide an indication of possible causes of the pavement failure.

Texture Depth of Road Surfacing

These tests measure the texture depth of the pavement. It may useful to know, since it is related to skid resistance, and may give information that will be useful when planning a surface treatment. Various methods are used including the sand patch method, and laser profilers.

Permeability of Surfacing and Granular Materials

These tests measure the permeability of the road surfacing and granular materials in-situ. If moisture is thought to be entering the pavement through the surfacing, it could be assessed using this test. An example of a device used to carry out this test is the Evenflow or Rapidflow Field Permeameter, used in Queensland.

Defect Survey and Mapping

These tests use a special vehicle to survey the road section and record all the defects. This enables a comprehensive record of defects to be compiled, which may prove useful in diagnosing the cause of failure.

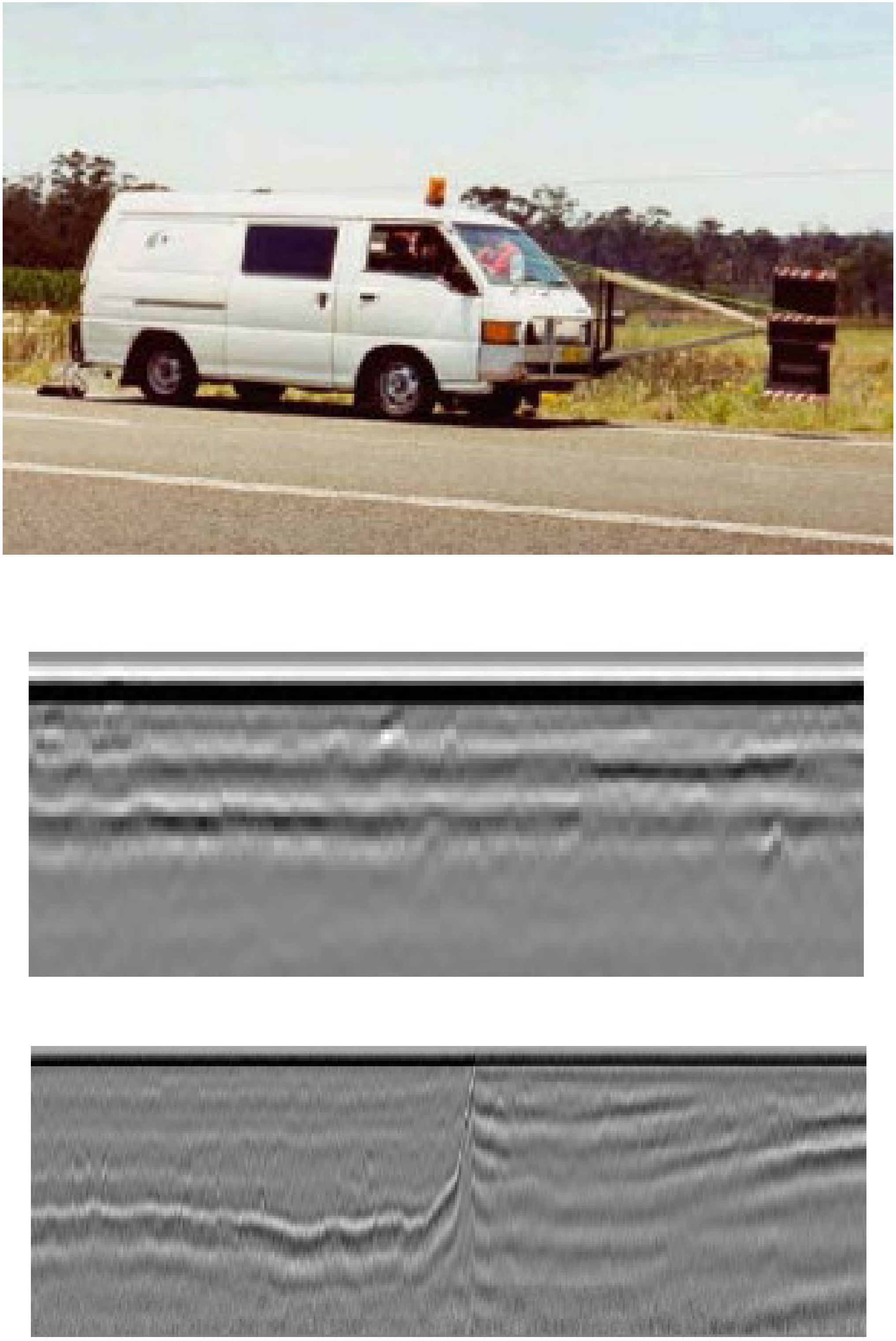


Figure 3.11 – Ground Penetrating Radar (GPR):

a) Van with air launched antenna at front, and ground coupled antenna at rear

b) A view of the interface between the various pavement layers

c) A view of the interface at a change in pavement construction and layer types

3.7 Destructive Materials Sampling and Testing

3.7.1 Introduction

This section discusses pavement testing that is destructive i.e. some of the pavement or underlying material must be taken away for testing, or disturbed in some other way. Generally, if samples are to be taken from a road, this must be done by digging a hole or trench, or by coring.

When testing materials to determine the cause of a failure, it is useful to test both failed and non-failed sections of road, so that comparisons can be made.

3.7.2 Guide to Materials Evaluation

A generalised guide to the evaluation of materials is given in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992). The two main aims of materials evaluation are as follows:

- Determine why the material has performed in a certain manner
- Obtain parameters for rehabilitation design

Granular Materials

It is important to identify various material types within the pavement structure and across its extent. Samples may be obtained from pits or trenches, or using drill probing. If the pavement has failed prematurely, close attention should be paid to the in-situ moisture contents, moisture gradient within layers, and the degree of saturation.

Stabilised Materials

The performance can only be measured in terms of the original design expectations, such as any strength gains expected from the stabilisation. It is important to check layer thicknesses and effectiveness of the stabilisation through the full depth.

Subgrade

The in-situ strength of the subgrade and existing pavement combination is often the limiting factor for rehabilitation work. Comparison of design subgrade CBR to in-situ values shows the extent of variation due to moisture, density and material changes.

The measurement of the subgrade moisture contents and variation along the pavement gives an indication of moisture ingress and the effectiveness of the moisture control/drainage system.

Asphalt and Bitumen Seals

The stability and stiffness of asphalt may be determined using Marshall testing, or some other method, and is strongly related to the rutting behaviour of the asphalt. Core samples or trenching may indicate whether rutting is occurring only within asphalt, or due to instability of lower layers, since subsurface profiles can be taken.

Test results from core samples may be compared to the relevant specification, or mix design test results. This may provide an indication of whether poor construction methods and/or materials were used. In addition to a lack of strength or excessive deformation, poor construction methods may often cause poor compaction, segregation or a reduced thickness of the layer.

An in-situ permeability testing procedure may be used to assess surface moisture entry. This could occur due to defects such as high voids, insufficient bitumen application, a ravelled surface, oxidised binder, stripped stone or cracking. Open Graded Asphalts should be placed over an impervious layer, with open transverse drainage on the low side of the layer

Problems with the binder viscosity may lead to flushing, stripping or ravelling. Standard procedures are available for testing viscosity. Test results may be compared to quality control testing to determine the remaining life of the bitumen, and if recycling is being used, the additional bitumen requirement may be assessed.

To assess whether degradation of the surface aggregate is occurring, it is important to compare the condition of stone in areas with and without traffic loading. If there is a notable difference, weathering, crushing, polishing or other tests may indicate if the aggregate is prone to degradation.

3.7.3 Assessing the need for Sampling and Testing

In many minor failures, destructive materials sampling and testing is often not done, and reliance is placed on non-destructive investigation and testing (such as a visual examination and/or deflection testing) to determine the cause of failure, and select the best rehabilitation treatment.

Destructive materials sampling and testing is costly due to costs of testing (personnel and laboratory costs), the need to repair any destruction of the pavement (such as filling in trenches) and the need for traffic control when sampling (in most cases). The point where this extra cost is justified by the results obtained is not easily defined.

Generally, the following must be considered:

- The scope of work as identified in the planning stage (and modified as required)
- The potential knowledge to be gained from testing, and its worth
- The cost of possible testing, relative to total cost of investigation, and possible rehabilitation treatments
- The potential for further costs if the rehabilitation treatment selected is a poor choice

When planning the investigation, the scope of work required would have been identified (and modified later as required). The factors influencing the scope of work must be considered when deciding on whether testing should be carried out, or not.

If a fair degree of certainty regarding failure cause has already been developed using previous steps in the investigation, destructive testing may not be required. However, if the initial investigation has failed to shed light on the problem, testing may be required, depending on the scope of work.

The cost of destructive testing and this cost relative to the total investigation cost and possible rehabilitation treatments must be considered. If this cost is relatively minor, then testing is likely to be cost-effective, whereas if the cost is a large proportion of the potential cost of a rehabilitation treatment, testing may not be justifiable on economic grounds.

The potential for extra costs if a poor rehabilitation treatment is selected must be considered. Depending on the failure magnitude, a (relatively) little cost of testing may translate into a long-term saving on maintenance costs, if the rehabilitation treatment selected is the best possible.

3.7.4 Trenching and Coring

Trenching and coring are the processes used to get samples of materials for testing, the methods of which are discussed below. Trenching is generally required for unbound materials, whereas coring may be used to get samples from bound materials.

As mentioned above, it is often useful to examine both unfailed and failed areas, so there is some point of comparison between them. Trenching and coring can also be used to give a visual inspection of the pavement structure. Important factors that should be noted are discussed below.

The strength of bond between layers of the pavement is important to observe. This includes the bond between the seal or surfacing and the base, and the bond between other pavement layers. Separation of the layers, or movement of the layers relative to the other may indicate poor bond.

A visual assessment should be made of the amount of moisture present in the layers. In addition, the change in moisture throughout a layer (moisture gradient) should be observed, and as well any accumulation of moisture at the interface between layers.

Figure 3.12 – A trench in a pavement, enabling the structure to be easily seen

Figure 3.13 – Coring of an asphalt sample

Other information that may be available from trenching or coring includes visual evidence of whether material breakdown is occurring in any layer(s), and subsurface profiles, typically used to provide an indication of which pavement layer(s) are undergoing deformation or rutting.

3.7.5 Introduction to Destructive Materials Testing

The following sections will describe some of the most common destructive testing methods available for the testing of pavement materials. It will cover both new performance-based tests that are increasingly being used, as well as older empirical tests still in common usage. It will include tests from the following categories.

- Soil and Aggregate
- Geotechnical
- Asphalt
- Bituminous Materials

The publication *Development of Performance-based Specifications for Unbound Granular Materials – Part A: Issues and Recommendations* (Austroads, 2003b) provides an overview of the multitude of tests and specifications used by these road agencies. These are currently almost all method-based, rather than performance–based specifications

3.7.6 Soil and Aggregate Tests

California Bearing Ratio (CBR)

The California Bearing Ratio test is commonly known throughout Australia as the CBR test, and may be done at standard or modified compactive effort, and at a nominated dry density and moisture content, or after soaking for several days.

In the test, a plunger is applied to a soil sample at a standard rate, and the loads to cause 2.5 and 5.0 mm penetrations are measured. The test relates the load vs. penetration curve of the selected material to that of a selected good quality fine crushed rock, and gives an indication of the stability or strength of the material.

The CBR value is widely used for pavement design, but is only really suitable for material under 20 mm nominal size. The type of test chosen is dependent on expected moisture conditions in the pavement, and the soaked CBR test is often used as it will give an indication of the stability of the material under extreme moisture conditions, such as flooding.

While there is now some indication that a CBR test is not the best possible means of determining the suitability of a material for use in a pavement (due to dissimilarity between a wheel loading on material and a plunger being pushed into the material), there is considerable knowledge about values obtained from the test and it will continue to be used, due to the relative ease of carrying out the test, and the widespread availability of the apparatus required.

Repeat Load Triaxial (RLT)

The Repeat Load Triaxial (RLT) test is only starting to be used in Australia, and is still uncommon. However, it will probably be used more in the future, since it measures fundamental material properties, and should provide a good representation of how a material will behave under traffic loading.

Due to the nature of the test, it will probably be more useful for the prevention of pavement failures by better characterisation of the materials used, rather than investigation once a failure has occurred. However, as the apparatus becomes cheaper and more widely used, it may be used for this purpose also.

It uses a similar test apparatus to a standard triaxial test, with a repeat load that simulates traffic loading being applied to the sample. Material properties that can be found from the test include resilient modulus, permanent strain rate, and the effect of varying stress, moisture and compaction on these properties.

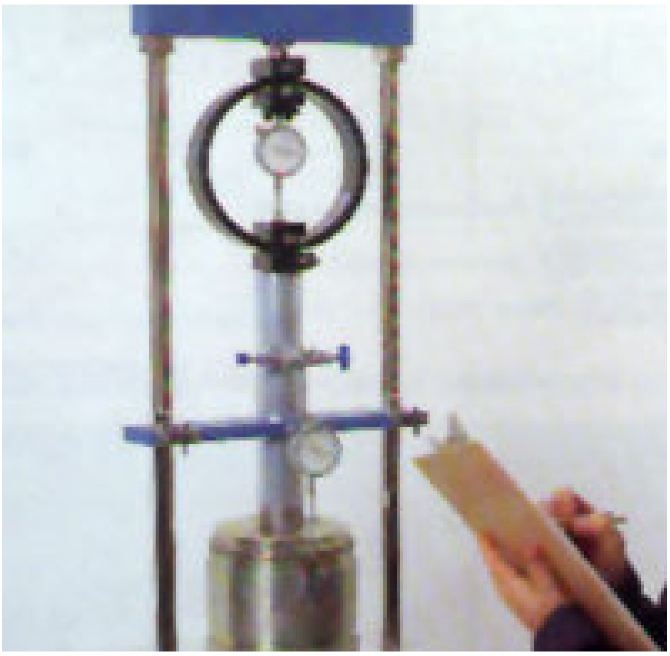


Figure 3.14 – The configuration used for the California Bearing Ratio (CBR) test

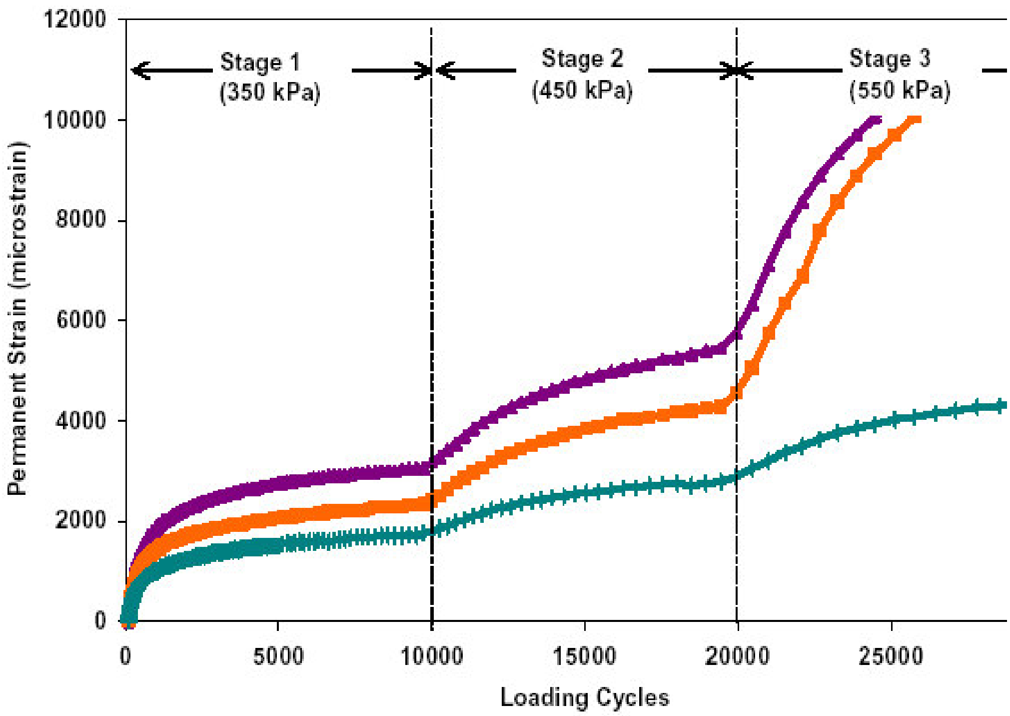


Figure 3.15 – A typical output from the Repeat Load Triaxial (RLT) test, showing that the higher the initial compaction (in green), the less the permanent strain after a number of loading cycles

Further information regarding the test can be found in the publication *Development of Performance-Based Specifications for Unbound Granular Materials - Part B: Use of RLT Test to Predict Performance* (Austroads, 2003c).

In-situ Strength

In-situ measurement of the strength of the pavement structure is most commonly achieved using a Dynamic Cone Penetrometer (DCP). A plot of depth of penetration vs. number of blows should be completed, from which sections with constant gradient can be seen (representing a soil layer).

For each soil layer, the results from testing may be converted to approximate CBR values using a formula or graph relating DCP penetration rate to CBR value. The correlation between DCP penetration rate and CBR value has been found empirically, and it is important to ensure that the formula or graph used is appropriate for the particular soil type.

It is also important to realise that the correlation is of limited accuracy, although this is to be expected. The publication *DCP Criteria for Performance Evaluation of Pavement Layers* (Gabr et al., 2000) provides more information about the relationship between DCP penetration rate and CBR values.

Advantages of this method include the speed and cost of testing, compared to CBR testing, for determining the in-situ strength. In addition, differing soil layers can be found.

Disadvantages include that it is only suitable for fine-grained soils and not very hard material, any seal must be removed before testing, and only the strength at the in-situ moisture content can be estimated. There is no ability to test the material at varying moisture contents, as with CBR testing. Taking account of these disadvantages, the DCP may be used in investigations to measure the strength of the material both cheaply and quickly.

Polishing

This form of testing is used for surfacing aggregates and generally conducted by using a pendulum tester to measure skid resistance after samples have been polished for a number of cycles by a rubber tyre in a special machine. It may indicate the susceptibility of an aggregate to polishing by traffic.

Crushing

These tests are commonly empirically based, and are used as a guide to how prone a surfacing aggregate or pavement material is to crushing under an applied load. They may also be used to provide a tentative guide to the strength and quality of the material. Since the tests are empirically based, it is difficult to relate the values found by using different test methods.

Common tests in this category include the 10% Fines Value, Aggregate Crushing Value, Los Angeles Abrasion, and Texas Ball Mill tests. The first two assess the resistance of the aggregate to crushing by the application of a steadily increasing load. The last two use a ball mill to test the abrasion qualities of the material

Soundness

These tests, empirically based, are used to assess the soundness of an aggregate, which is a general measure of the durability of the material when exposed to attack by chemical or physical conditions. A variety of tests are used, and it is difficult to relate the resulting values to each other, due to the different testing methods and conditions used.

The most common test used to assess the soundness of an aggregate is conducted using a sodium sulphate solution (other chemicals may also occasionally be used). In this test, single sized portions of the aggregate are subjected to cycles of immersion in sodium sulphate solution and drying. After the specified number of cycles, the % of fine particles (below a certain size) is found. This is repeated for various sizes within the aggregate grading, and the overall soundness of the aggregate found from this.

This test may give an indication of the susceptibility of the aggregate to chemical attack by sodium sulphate. Various other empirical tests may also be used, including weak particle tests, and degradation factor tests.

Moisture Content

This commonly used test measures the moisture content of the soil (Mass of Water relative to Mass of Solids) using oven, microwave, sand bath, hotplate, or infrared lights drying. A moisture content test may indicate whether excess moisture is in the pavement. For this reason, the test would be an important one in any failure where moisture is suspected as being a contributing factor.

When testing, it may not be suitable to measure the average moisture content in the pavement, since the averaging out effect may mean that high moisture contents at either the top or bottom of the pavement are not shown by the test results. The more layers that are considered separately when calculating moisture content, the more accurate the testing will be in giving an indication of the moisture gradient within the pavement.

Degree of Saturation

This test, used mostly in Queensland, measures the degree of saturation of the soil (volume of water relative to volume of void space). This can be calculated from moisture content, dry density and particle density.

This property may be even more important than the moisture content in determining whether excess moisture is causing failures, since a material may have a moderate moisture content, but a high degree of saturation.

A high degree of saturation may cause significant pore pressures to develop, and traffic loadings may pump water in porous pavements, especially crushed rock. This may lead to a rapid shear/bearing failure, rutting due to reduced stiffness, lifting of surfacing course due to positive pore pressure, or embedment of cover aggregate due to weak base. For these reasons, degree of saturation should be kept low (typically below 60-70%), to help ensure that these effects do not occur.

Figure 3.16 – Soil samples being put in an oven, as part of moisture content testing

Figure 3.17 – Sieves as used to measure the particle size distribution of a soil sample

Particle Size Distribution

This common form of testing measures the grading of the material, and records the percentage of material passing certain sieve sizes. It may be done using either wet or dry sieving, or with a hydrometer for very fine particles. A Particle Size Distribution test may indicate whether crushing of material is occurring or a problem with the material being produced being outside the specified limits.

A typical grading curve is shown (along with specified grading limits, as would be required by the relevant specification). Sieve size is often plotted as a logarithmic scale.

When testing is done with a large layer thickness, it may fail to reveal a problem in one specific depth range, due to averaging (similar to moisture content testing). For example, a grading test of a 250 mm thick layer may fail to reveal that the top 50 mm of material is breaking down. For this reason, testing separately over smaller depths is recommended.

Particle Shape

These tests are used to assess the shape of surfacing aggregates or pavement materials, using a variety of methods, some of which are discussed.

Flakiness index is a common test used to measure the % of flaky particles in a sample. A flaky particle is one with least dimension less than 0.6 the mean dimension (mean of passing and retained sieve sizes). Testing is done to determine the % of flaky particles in each single-sized fraction (separated by sieving), and the flakiness index is the % of flaky particles overall.

The average least dimension test measures the average least dimension (ALD) of the particles in a sample of surfacing aggregate. This is an important parameter for spray seal design and the accurate measurement of this value is important to help ensure the correct bitumen application rate is used when sealing.

Figure 3.18 – Sieves attached to a shaker that speeds up the grading test process

Figure 3.19 – Assessing the grading of fine particles using a hydrometer

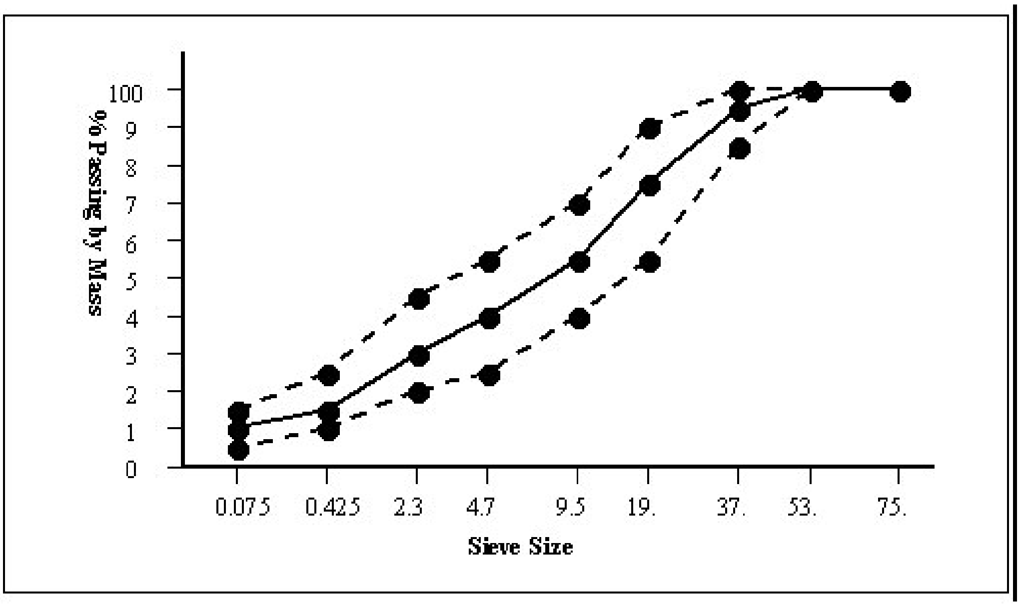


Figure 3.20 – Typical Particle Size Distribution (Grading) Curve, with Specification Limits

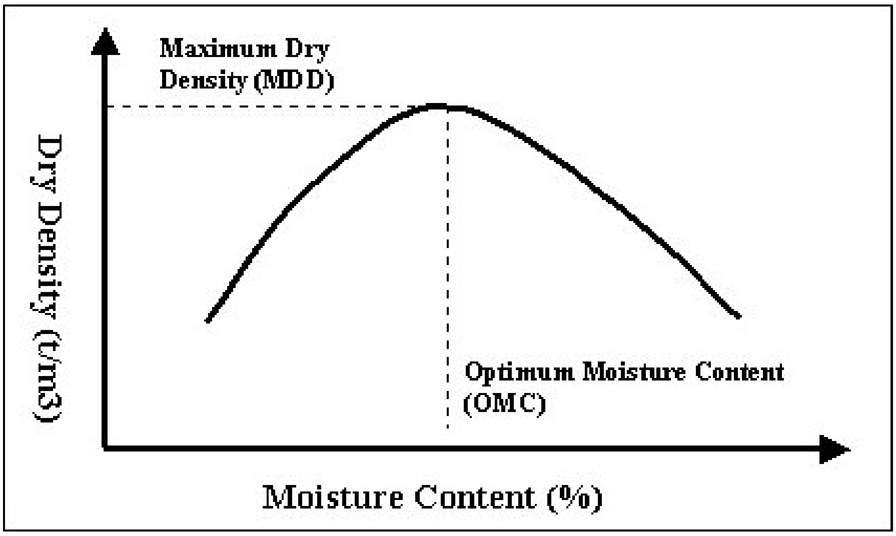


Figure 3.21 – A typical Relationship between Dry Density and Moisture Content for a soil sample

Figure 3.22 – Measurement of soil density:

a) A Nuclear Gauge

b) A sand cone apparatus

Dry Density

The dry density (and also moisture content) of an in-situ soil may be measured using the sand replacement, water balloon or nuclear gauge methods. The dry density is calculated from the bulk density of the sample using the moisture content.

Samples of the material may be taken and tested at a standard compactive effort and varying moisture contents. The relationship between dry density and moisture content is usually similar to that shown in the Figure 3.21.

Once this testing is complete, the optimum moisture content (OMC) and maximum dry density (MDD) can be found. The relative density of the measured in-situ dry density can be found by comparing it to maximum dry density found from the testing. Low relative density may indicate poor compaction.

For a cohesionless material, the procedure is different, with the in-situ density of the material being compared to the minimum and maximum dry densities, allowing the density index to be calculated.

Other tests may be used to measure the particle density of the soil or aggregate. This density is that of the particles, and does not include moisture or air voids.

Atterberg Limits

These empirical tests are used to find the moisture content at which a soil changes from one state to another e.g. from liquid to plastic. These limits are arbitrarily defined by the tests, since an exact determination of these points is not really possible. The most common Atterberg limits tests used are discussed.

The liquid limit (LL) is the lowest moisture content at which the soil behaves in a liquid manner. This is determined in an arbitrary manner, most commonly by measuring the penetration of a cone penetrometer into the soil sample at varying moisture contents. It may also be measured using the Casagrande apparatus.

The plastic limit (PL) is the lowest moisture content at which the soil behaves in a plastic manner. This is determined in an arbitrary manner by rolling soil samples into ribbons.

The plasticity index (PI) is defined as the difference between the liquid limit and the plastic limit. These above values give an indication of the amount of clay present in the material, and the potential for activity of the clay present.

Linear Shrinkage

This test measures the percentage shrinkage of the fine fraction of the material after drying from wet condition. It gives an indication of the volume change likely to occur when the moisture content changes. The test is often done in conjunction with the Atterberg limits testing, being very easy to carry out.

Miscellaneous

Other tests may be used for the testing of soil and aggregates as required by the particular circumstance. The tests available vary widely depending on the Road Agency, and so are not listed here.

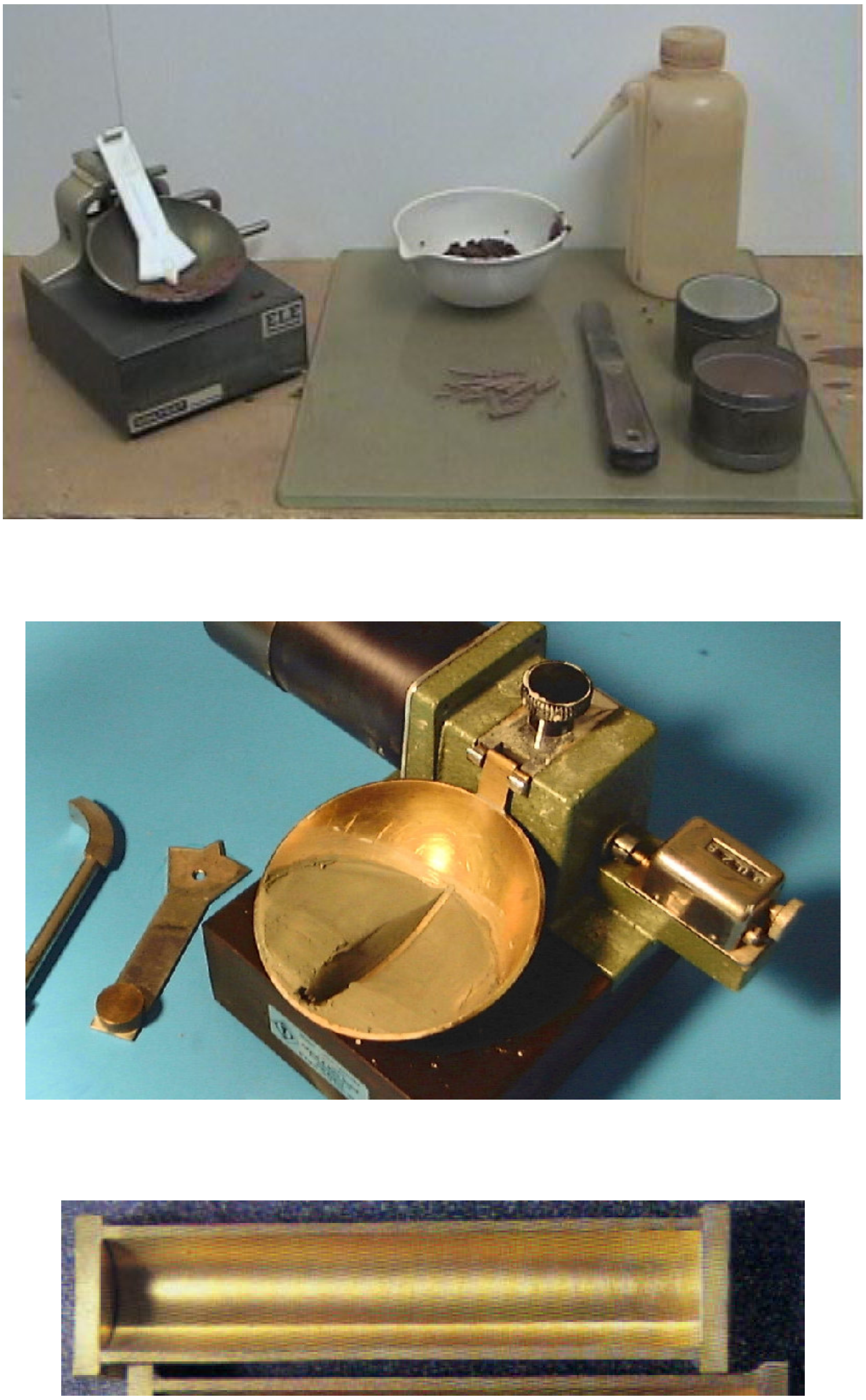


Figure 3.23 – Equipment commonly used for measuring other soil properties:

a) Equipment used for measuring the Atterberg Limits

b) The Casagrande Apparatus, sometimes used to measure the Liquid Limit

c) A mould used to measure the Linear Shrinkage of a soil sample

3.7.7 Geotechnical Tests

Penetration

These assess material properties using a penetration device of some kind. Types may include bridge probes, electric friction cone penetrometer, field shear vane, and standard penetration tests.

Water Table Measurement

This measures the level of a water table, often using drill holes. This information may indicate whether a high water table is causing excess moisture to move into the pavement.

Compressive Strength

This test measures the compressive strength of a sample, often in an unconfined condition. It may be of limited importance since soil is normally confined, but is may be useful for the assessment of stabilised materials.

Triaxial

This testing has many different forms. The choice of which test is best to use is dependant on the site conditions. Repeat Load Triaxial (RLT) testing was discussed above, but other triaxial tests include unconsolidated undrained, consolidated undrained, and consolidated drained.

Consolidation

This testing assesses the consolidation of a soil sample. It may give an indication of the amount of consolidation that would be expected under loading. Testing is often carried out using an oedometer.

Shear Strength

This testing assesses the angle of internal friction of the soil sample (which is a measure of shear strength), often using a shear box apparatus. This testing is normally used for cohesionless soils such as sand, since for cohesive soils, the values of internal friction angle and cohesion can normally be found from triaxial testing.

3.7.8 Asphalt Tests

Mix Design of Asphalt

These testing methods are used for the mix design of asphalt, by measuring the stability (maximum load that can be carried) and flow (deformation of sample) found for the standard conditions used in the test.

In these empirical tests, varying binder contents are used for a number of samples and measuring the stability and flow of each can enable the optimal binder content to be selected. A lack of stability, flow or stiffness (stability/flow) may be a cause of problems in the asphalt.

Test methods include the Marshall method, used in Queensland, which uses impact compaction techniques. However, since this is dissimilar to the vibratory compaction techniques used in the placement of real asphalt, other procedures have been developed that allow asphalt stiffness and deformation properties to be tested, while better simulating the actual heating and compaction process used in the placement of asphalt.

The choice of which testing method to use would be strongly dependent upon the testing apparatus, expertise and preferences of the particular road agency.

Deformation of Asphalt

One test used for the measurement of asphalt deformation is the wheel tracker test, used to estimate the possible rutting potential of an asphalt sample. While there are a wide variety of conditions used, in all a test sample is subjected to a wheel being run across the top of it for a number of cycles. The test may also be used for granular materials or aggregates as needed.

Properties that can be found from the test include the rutting profile, rutting rate and final rut depth. This test simulates the actual traffic-loading situation where wheels run across the pavement, and therefore could be expected to give a good indication of the expected rutting behaviour under traffic.

In addition, the influence of the moisture content or degree of saturation on the performance of the material could be examined by testing the material at varying degrees, and comparing the results. It should be noted however, that like any empirical test, it is difficult to use the values for anything outside the particular test, and a range of typical values are required to enable the test results to be used correctly.

Other

Other tests used for asphalt include those for the measurement of sensitivity to water, susceptibility to abrasion loss, binder content, aggregate grading, compacted relative density, and air voids calculations. Generally, the grading and density tests are similar to those for soils and aggregates, while taking account of the different materials being tested.

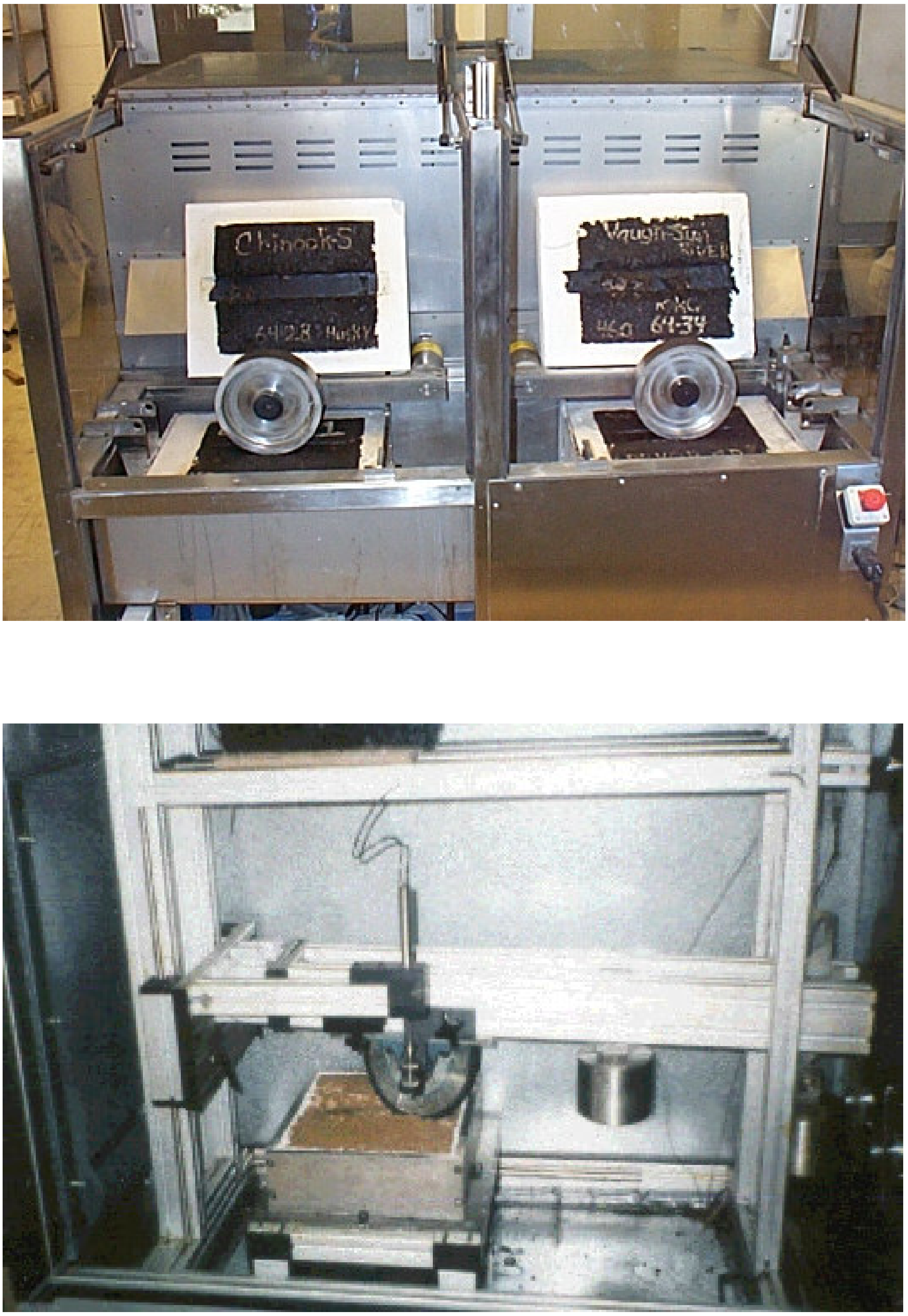


Figure 3.24 – A view of a wheel tracker testing:

a) Used on asphalt

b) Used on granular material

Figure 3.25 – Asphalt testing:

a) The Marshall testing apparatus, commonly used for the mix design of asphalt

b) A gyratory compactor, which simulates the compaction used in actual asphalt production

3.7.9 Bituminous Material Tests

The testing of bituminous materials may be split into those tests that are required for all bitumen, and other tests that are required for polymer modified binders, which require additional properties to be successful in their intended application.

The rolling thin film oven (RTFO) test is used to determine the susceptibility of the bitumen to heat and air, measuring ductility and viscosity after oven testing. This is intended to give an idea of the degree of bitumen hardening that will occur throughout its life.

Density may be measured using several types of tests, including the density bottle or hydrometer methods. Viscosity (shear resistance at a particular temperature) can be measured using one of several types of viscometer.

Solubility of bitumen is a measurement of the purity of the bitumen sample. A lack of purity may indicate quality control problems. Flash point tests are used to find the temperature at which bitumen will ignite. It is primarily used to specify the safety point to which bitumen may be heated.

The softening point of bitumen is measured since excessive softening at some temperatures will cause problems. For example, in a hot climate, excessive softening at a low temperature would be undesirable.

The penetration of bitumen is specified, by measuring the indentation of a needle of specific dimensions falling into a bitumen sample over a certain time, at a particular temperature. This gives an indication of the hardness of the bitumen at a particular temperature.

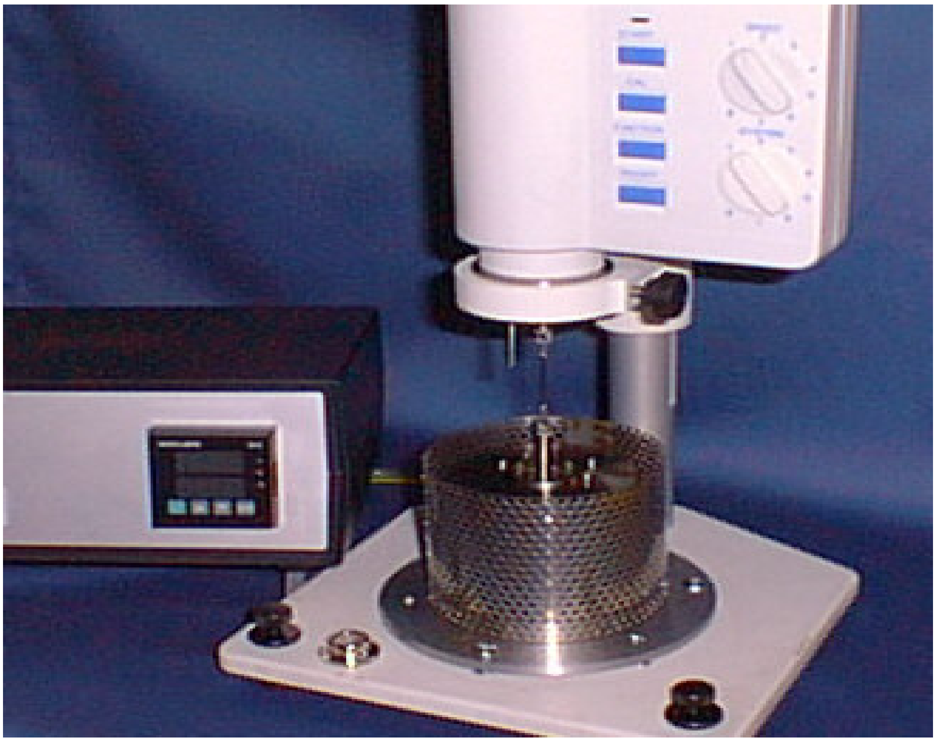


Figure 3.26 – Bituminous material tests:

a) A Rolling Thin Film Oven (RTFO) used to estimate the long-term performance of a bitumen sample

b) A rotational viscometer, one of several apparatus used to determine the viscosity

3.7.10 Tests used in Queensland Main Roads

This section gives details about what tests are included in the Queensland Department of Main Roads Standard Specifications (related to pavements).

This should provide an indication of what tests are currently used by a typical Australian Road Agency. It should be noted that the specifications are continually changing to reflect the increased knowledge of material behaviour and properties.

Table 3.12 – Tests in MRS 11.04 – General Earthworks

Soil and Aggregate Tests: Strength and Durability	California Bearing Ratio (also in-situ)
Soil and Aggregate Tests: Particle Size and Shape	Particle Size Distribution
Soil and Aggregate Tests: Density	Dry Density Measurement (sand, nuclear gauge) Dry Density–Moisture Content relationship Density Index of a Cohesionless Material
Soil and Aggregate Tests: Other Material Properties	Liquid Limit, Plastic Limit Plasticity Index, Linear Shrinkage pH Value, Electrical resistivity, Chloride Content, Sulphate Content, Lime Demand

Table 3.13 – Tests in MRS 11.05 - Unbound Pavements

Soil and Aggregate Tests: Strength and Durability	CBR Values (Soaked) Ten Percent Fines Value, Wet/Dry Strength Variation Degradation Factor, Crushed Particles
Soil and Aggregate Tests: Moisture	Moisture Content Degree of Saturation
Soil and Aggregate Tests: Particle Size and Shape	Particle Size Distribution Flakiness Index
Soil and Aggregate Tests: Density	Dry Density Measurement (sand, nuclear gauge) Dry Density–Moisture Content relationship Density Index of a Cohesionless Material
Soil and Aggregate Tests: Other Material Properties	Liquid Limit, Plastic Limit, Plasticity Index Linear Shrinkage

Table 3.14 – Tests in MRS 11.17 - Bitumen

Bituminous Material Tests: Bitumen	Dynamic Viscosity (several methods) Density, Solubility Flash Point, Penetration Rolling Thin Film Oven (RTFO) Test

Table 3.15 – Tests in MRS 11.18 – Polymer Modified Binder

Bituminous Material Tests: Polymer Modified Binder	Flash Point, Softening Point Elastic Recovery, Consistency Stiffness (Elastometer), Torsional Recovery Viscosity, Handling and Preparation Segregation, Ease of Remixing Toughness (Extensiometer), Compression Limit Loss on heating (Rolling Thin Film Oven test)

Table 3.16 – Tests in MRS 11.22 – Supply of Cover Aggregate

Soil and Aggregate Tests: Strength and Durability	Ten Percent Fines Value, Wet/Dry Strength Variation Crushed Particles, Weak Particles
Soil and Aggregate Tests: Particle Size and Shape	Particle Size Distribution of Aggregate Flakiness Index
Soil and Aggregate Tests: Density	In-situ Dry Density (Sand Replacement) Dry Density – Moisture Content Relationship
Soil and Aggregate Tests: Other Material Properties	Degree of Aggregate Precoating

Table 3.17 – Tests in MRS 11.30 – Dense Graded Asphalt Pavements

Soil and Aggregate Tests: Strength and Durability	Polished Aggregate Friction Value Ten Percent Fines Value Wet/Dry Strength Variation Degradation Factor Crushed Particles Weak Particles
Soil and Aggregate Tests: Particle Size and Shape	Particle Size Distribution Flakiness Index Particle Density and Water Absorption
Asphalt Tests: Strength and Durability	Stability and Flow of Asphalt (Marshall)
Asphalt Tests: Other	Dry/Wet Coring of Bound Materials Preparation of Asphalt Core Samples and Mix Compacted Density of Asphalt Binder Content, Aggregate Grading of Asphalt Preparation and Testing of Tolerance Mixes Voids Calculation for Compacted Asphalt Relative Compaction of Asphalt Voids in dry compacted filler

3.8 Determine Probable Cause(s) of Failure

3.8.1 Introduction

This section is perhaps the most difficult section to carry out in the investigation, since there are often uncertainties and multiple contributing factors to a pavement failure. A procedure that may enable the probable cause(s) of failure to be determined is as follows:

- Assemble all information from the investigation in a logical and coherent way.

- Determine what information there is to support/refute each of the possible failure hypotheses.

- Determine which cause(s) of failure is most likely, based on this.

This section should be used as a guide, not only when trying to determine the cause(s) of the failure, but throughout the investigation as it can also help to provide information about visual investigation or testing that may need to be carried out for the specific failure type.

3.8.2 Investigative Synthesis

In the latter stages of the investigation, when much investigation and testing has been done, and the probable cause of failure is being considered, all the information that has been obtained must be linked together in a logical and coherent way, to ensure no important details have been missed.

The simplest way to do this is to prepare a report listing details about the project, inspection and testing carried out and their results. All information obtained should be listed, and feedback from other team members will ensure nothing important is missed.

3.8.3 Determine evidence for each Failure Hypotheses

The simplest way to do this is to go through a list of possible causes of failure, and determine what evidence there is to support each hypothesis. This may provide an indication of the failure cause. First general causes of failure will be discussed, followed by a more detailed explanation of specific causes of failure. Possible sources for this information are also listed.

General Causes of Failure

Construction Problems

There may have be insufficient compliance with the relevant specifications, or an incompatibility between design, materials and construction.

There also may have been incorrect curing conditions or exposure to traffic, inexperienced personnel or inadequate testing and inspection. There may have also been a particular problem during construction that may have contributed to the failure.

Information about these possible problems may be found from reviewing documents and literature, interviewing personnel, and sampling and testing of materials.

Materials Problems

There could have been a large variability of materials or their properties, including larger than expected changes in material properties over time due to moisture and temperature.

Material function and location may have varied along the road length, leading to incorrect placement. There also could have been improper testing or blending of materials. In some cases, materials are incompatible and should not be used together.

Information about these possible problems may be found from reviewing documents and literature, interviewing personnel, and sampling and testing of materials.

Design Problems

There may have been insufficient site inspection prior to design, and hence insufficient information about the pavement and its existing condition. This may have led to constraints on the design not having been understood or incorporated.

There could have been poor design of drainage structures, insufficient projection of traffic volumes, not enough attention paid to material design in the design process, or a design constrained by a lack of funds.

Information about these possible problems may be found from reviewing documents and literature and interviewing personnel.

Environment Problems

These can be widely varied, but high moisture content or changes in it are often a problem. There may have been chemical conditions that contributed to the failure, or some other environmental aspect that was overlooked or not anticipated.

Information about these possible problems may be found from onsite visual investigation and materials sampling and testing, especially specialised chemical testing as required.

Specific Causes of Failure

This section lists important possible failure causes for each failure type, shown in table format. Considering these may reveal what the probable cause(s) of failure was. Possible sources of the information are listed as required.

Table 3.18 – Failure causes and information sources for bleeding and flushing

Possible Failure Cause	**Possible Information Source**
Too much binder sprayed	Seal design calculations, actual spray rate Construction personnel
Insufficient surface aggregate applied	Seal design calculations, actual rates Construction personnel, visual inspection
Non-uniformity/patching of original surfacing, leading to rise of binder	Any information about patching work Area inspector / engineer Visual inspection of non-uniformity
Embedment of surface aggregate, due to weakness of base layer below	Visual inspection of weak base or moisture entry, deflection testing Strength / stability / moisture of base
Lack of proper rolling during placement	Rolling during construction and effectiveness Construction personnel
Failure to protect newly constructed surface from traffic for long enough	Traffic control information Construction personnel
Loss of surface aggregate due to stripping or ravelling	Visual inspection of stripping or ravelling
Breakdown of surface aggregate	Visual inspection of breakdown Grading / strength / crushing
Poor spreading of aggregate	Spread rate and uniformity Construction personnel Visual inspection of poor spreading
Over-filled voids in asphalt	Mix design of asphalt Visual inspection of over-filled voids Voids content
Lack of size of aggregate leading to being covered by binder	Original reports on grading / particle size

Table 3.19 – Failure causes and information sources for cracking

Possible Failure Cause	Possible Information Source
Cracking due to ageing and embrittlement of surfacing	
Old Seal	Seal age
Degradation of binder due to other influences	Visual inspection Testing
Block Cracking	
Inability of binder to expand and contract, due to aged binder	Seal age Visual inspection
Inability of binder to expand and contract, due to stiff binder	Binder testing Binder testing
Reflection Cracking	
Caused by horizontal or vertical movements of pavement below overlay due to temperature or moisture changes	Evidence of cracked pavement underneath Visual inspection Testing
Shrinkage Cracking	
Volume change within surfacing, base or subgrade	Past evidence Visual inspection Shrinkage / plasticity
Cracking due to structural failure	
Fatigue failure of one or more pavement layers	Age of pavement Visual inspection Test results
Excessive moisture or poor drainage	Visual inspection Moisture
Permanent severe deformation of the subgrade due to repetitive loading	Loading on pavement Visual inspection Testing

Possible Failure Cause	**Possible Information Source**
Instability in upper pavement layers	Visual inspection Strength / stability / moisture
Repeated deflection causing fatigue in the surfacing	Deflection testing Testing
Inadequate pavement thickness	Pavement design
Lack of stiffness in base	Visual inspection, Deflection Testing Strength / Stability
Increase in traffic loading, not accounted for in design	Pavement design, Traffic records
Poor construction, including poor compaction	Construction records Construction personnel Visual inspection
Joint Cracking	
Weak seams of adjoining pavement spreads in the surface layers	Visual inspection
Wetting or drying action beneath shoulder surface caused by trapped water standing and seeping through joint between shoulder and mainline pavement	Visual inspection Moisture
Longitudinal Cracking	
Poor joint construction or location	Construction records Construction personnel Visual inspection
Reflective cracking from below	Evidence or cracked pavement underneath Visual inspection
Onset of structural failure, leading to crocodile cracking	Pavement and seal age Visual inspection

Possible Failure Cause	Possible Information Source
Slippage Cracking	
Insufficient bond between the surface and underlaying courses, caused by dust, oil, rubber, dirt, water	Visual inspection Trenching: bond
No tack coat between the two courses	Construction records Construction personnel
Transverse Cracking	
The result of reflection cracking	Evidence of cracked pavement underneath Visual inspection
Stresses induced by low-temperature contraction of the pavement, especially if the asphalt is hard and brittle	Mix design and previous use, Past temperatures Material s officer Testing: hardness / brittleness

Table 3.20 – Failure causes and information sources for corrugations

Possible Failure Cause	Possible Information Source
Inadequate stability of asphalt surfacing or base under traffic	Mix design, compaction, strength / stability Visual inspection of lack of stability or moisture Strength / stability / moisture
Non-uniform compaction of the surfacing or base during construction	Compaction and variation Visual inspection of variance Evidence of variance in testing

Table 3.21 – Failure causes and information sources for depressions

Possible Failure Cause	Possible Information Source
Settlement of embankments (e.g. near bridge abutments)	Visual inspection
Poorly compacted surface trenches	Visual inspection
Moisture weakening pavement	Visual inspection Moisture / strength

Table 3.22 – Failure causes and information sources for edge defects

Possible Failure Cause	Possible Information Source
Fretting	
Lack of support due to weakening of the pavement near edge	Visual inspection Strength / stability / moisture
Worn shoulders, giving insufficient side support to pavement	Visual inspection
Traffic running too close to edge (narrow lanes, tracking)	Visual inspection
Worsened by presence of water	Visual inspection Moisture
Drop-off	
Traffic running off the surfacing and abrading shoulder material	Visual inspection
Inadequate lane width or tracking?	Visual inspection
Worsened by presence of water	Visual inspection Moisture

Table 3.23 – Failure causes and information sources for patching

Possible Failure Cause	Possible Information Source
Localised remedial work in response to some form of surface distress	Maintenance Visual inspection: reason for patching
Many recent patches indicating an ongoing problem requiring overall retreatment	Visual inspection: reason for patching Testing

Table 3.24 – Failure causes and information sources for polishing

Possible Failure Cause	Possible Information Source
Some aggregates including limestones becoming polished and slick, especially when wet	Past evidence of polishing in material Visual inspection Polishing

Table 3.25 – Failure causes and information sources for potholes

Possible Failure Cause	Possible Information Source
Inadequate pavement or aged surfacing	Age of seal and pavement Visual inspection, Deflection testing Strength
Moisture entry into pavement	Visual inspection Moisture / strength
Cracked surface	Visual inspection
High shoulders or pavement depressions ponding water on the pavement	Visual inspection
Porous or open surface	Visual inspection Voids / permeability
Clogged side ditches	Visual inspection

Table 3.26 – Failure causes and information sources for rutting

Possible Failure Cause	Possible Information Source
Inadequate pavement thickness	Pavement Design
Weak subgrade (adjacent bulging of pavement or shoulder surface)	Design strength and basis Visual inspection, Deflection Testing Strength / moisture
Weak base	Visual inspection, Deflection testing Strength
Surfacing lack of strength / stability	Visual inspection, Deflection testing Strength / stability
Inadequate Compaction	Compaction during construction Compaction testing
Poor Material Quality	Evidence Construction personnel / materials officer Testing
Excessive Moisture	Visual inspection Moisture
Very high traffic loading (> 10^7 ESAs)	Traffic volumes and loading Design personnel
Inappropriate mix design (asphalt): - Too much binder - Too much filler - Insufficient angular aggregates	Mix design and past use Construction personnel / materials officer Mix design
Interaction between layers (e.g. excess cutter moving into next layer)?	Evidence due to construction procedure Construction personnel Visual inspection Testing

Table 3.27 – Failure causes and information sources for shoving

Possible Failure Cause	Possible Information Source
Inadequate strength/stability in surfacing or base material	Testing during construction Visual inspection Strength / stability
Poor bond between pavement layers	Visual inspection Trenching: bond between layers
Lack of containment of pavement edge	Visual inspection
Inadequate pavement thickness overstressing the subgrade	Pavement design Testing
Excessive moisture	Visual inspection Moisture
Contamination caused by oil spillage	Written evidence Personnel Visual inspection
Lack of curing time between placing seal treatments	Construction procedure Construction personnel
A mixture that is too rich in asphalt	Mix design and use in past Visual inspection of excess asphalt Asphalt content, strength / stability
Asphalt has too much fine aggregate	Mix design and use past Asphalt grading
Asphalt has too much aggregate that is rounded or smooth-textured	Material and use in past Strength / stability, particle shape
Asphalt cement is too soft	Asphalt, high temperatures Hardness of asphalt

Table 3.28 – Failure causes and information sources for stripping and ravelling

Possible Failure Cause	Possible Information Source
Inadequate binder application	Seal design, actual rate Mix design Construction personnel Visual inspection
Lack of bond between aggregate and binder, due to lack of precoating or adhesion agents, or dirty, dusty or soft aggregate	Evidence of problems in past, construction records, condition of aggregate Construction personnel / materials officer Evidence of breakdown Stripping testing, breakdown / softness
Degradation of binder through ageing or traffic damage, solvent or chemical spillage	Written evidence Personnel Visual inspection
Incompatibility between aggregate rock type and binder	Evidence of problems in past Construction personnel / materials officer Testing
Insufficient/excessive cutter in binder when sprayed, leading to reduced wetting or too soft to hold aggregate under traffic	Construction records Construction personnel
Excessively open-graded asphalt mix	Mix design and use Construction personnel / materials officer Voids, mix design
Insufficient blending of binder before spraying	Construction records Construction personnel
Poor mix design	Similar use in past Materials officer Mix design
Aggregate segregation	Construction personnel

Possible Failure Cause	Possible Information Source
Lack of compaction of surface during construction	Compaction Construction personnel Compaction
Insufficient rolling / excessive brooming before exposure to traffic	Construction records, rolling Construction personnel
Aggregate size incompatible with that of previous seal	Evidence of similar problems, guidelines Construction personnel Visual inspection Testing
Fracturing of aggregate	Evidence of problems in past Construction personnel / materials officer Visual inspection Testing: breakdown / grading
Hydrophilic aggregate	Materials officer Testing
A dry brittle surface	Construction records Construction personnel
Patching beyond base material	Patching work Visual inspection: non-uniformity of stripping
Excessive heating during mixing	Construction Records Construction personnel
Construction during wet or cold weather	Construction records Construction personnel

3.8.4 Determine Probable Cause(s) of Failure

Once a review has been made of the information available about each of the possible failure causes, the most probable cause(s) must be decided upon. Two possible methods of doing this are discussed below.

Matrix Method

This method of presentation, suggested by Crampton et al. (2001), gives a systematic method of reviewing the data, and may highlight relationships that are not present when displayed in any other way.

Down the first column, objective information such as testing results and visual observations are listed. Across the first row, subjective information such as possible failure causes is listed.

Table 3.29 – Sample matrix of facts and failure hypotheses for bleeding and flushing

Y = Fact supports hypotheses N = Fact refutes hypotheses		Possible failure causes			
		Too much binder	Not enough aggregate	Embedment of aggregate (Weak base)	Breakdown of aggregate
Objective Information (facts)	Located in wheelpaths, with little or none outside			Y	Y
	No lack of aggregate applied during sealing		N		
	No evidence of excess binder being sprayed	N			
	No evidence of aggregate breakdown				N
	Base appears to be moist, with poor drainage			Y	

In the cells, a mark should be made to indicate whether the objective information supports or refutes the possible failure cause. Inconclusive information should not be included, although the determination of whether information is inconclusive or not is very subjective. A simplified example of the matrix method is shown in the table.

From the table, it can be seen that because the bleeding and flushing is mostly located in the wheelpaths and there is evidence that the base is moist due to poor drainage, the probable cause of the failure is embedment of the surface aggregate into the base below, due to moisture entry resulting from the poor drainage.

Weighted Matrix Method

Table 3.30 – Sample weighted matrix of facts and failure hypotheses for bleeding and flushing

-1: Disagreement -0.5: Some disagreement 0: Neutral 0.5: Some agreement 1: Agreement		Possible failure causes			
		Too much binder	Not enough aggregate	Embedment of aggregate (weak base)	Breakdown of aggregate
Objective Information (facts)	Located in wheelpaths, with little or none outside			1	0.5
	No lack of aggregate applied during sealing		- 0.5		
	No evidence of excess binder being sprayed	- 0.5			
	No evidence of aggregate breakdown				-1
	Base appears to be moist, with poor drainage			0.5	
		-0.5	**-0.5**	**1.5**	**-0.5**

A more complicated variation on the matrix method is the weighted matrix method. In this, instead of using a simple Y(es) or N(o), a weighting is used, ranging from 0 to 1. The meanings of the numbers may be as follows:

- -1: Fact refutes hypotheses (disagreement)
- -0.5: Fact somewhat refutes hypotheses (some disagreement)
- 0: Fact neither refutes nor supports hypotheses (neutral)
- 0.5: Fact somewhat supports hypotheses (some agreement)
- 1: Facts supports hypotheses (agreement)

To determine the probable failure cause, the sum for each failure cause is determined. The cause with the highest sum is probably the most likely failure cause, although judgment should still be used when making a decision. A simplified example of the weighted matrix method is shown (with zero values being left blank).

3.8.5 Learning from the Failure

Once the probable cause(s) of failure have been determined, the information and knowledge learned during the investigation should be listed, for inclusion in the final report.

This should include any unusual aspects of the failure, and also any recommendations as to how the failure could be prevented in the future, through improved design, construction or maintenance practices, or through an increased knowledge of the materials used and their properties.

3.9 Determine Best Rehabilitation Treatment

3.9.1 Introduction

The investigation of a pavement failure should reveal the probable cause(s) of the failure. Once the failure cause(s) is known, the best rehabilitation treatment can be selected. This selection process is very important since choosing the best rehabilitation treatment will mean the road will last longer before deteriorating again, and provide the best conditions for road users.

The best rehabilitation treatment is that which will cost least over the life cycle of the road. This is not necessarily the cheapest option initially, but is dependent on the conditions that caused the initial failure of the pavement.

It should be recognised that some factors, such as available plant and materials and cost limitations, may mean that the best option cannot be used, and instead the best possible treatment should be selected.

This section will discuss the process used in selecting a rehabilitation treatment, and factors that must be considered, as detailed in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992).

A brief description of the rehabilitation options presented in the *Pavement Rehabilitation Manual* and newer rehabilitation options will be provided. For more information, the reader should refer to this, or other more up to date publications as relevant.

3.9.2 Selection of the best rehabilitation treatment

To select the best possible rehabilitation treatment, firstly the pavement type should be identified, and then the purpose of the rehabilitation treatment decided upon. From the *Guide to Selection of Road Surfacings* (Austroads, 2000a), factors to consider may include the following that are discussed below.

The location of the failure is important. Some treatments are best for urban areas, whereas others are best for rural areas. Noise may be an important issue in urban areas. If the location is in a particularly stressful area such as on a steep grade or near an intersection, then some treatments are preferable to others. Traffic volumes must be considered, as well as the effects of any traffic disruption.

The roughness and the need for shape correction should be considered. For example, whether any structural strengthening or rut correction works must be done. In addition, the need for the repair of cracking should be looked at.

The state of the existing road surface should be examined. In particular, how oxidised it is, whether surface texture is a problem, and the need for an improvement in skid resistance. Surface drainage, the uniformity of the surfacing, and the extent of bleeding/flushing of surface should also be considered.

Other factors to consider include whether the treatment is accepted local practice, and whether a long lasting or simply an economical short-term treatment is required. Once these questions are answered, a list of possible rehabilitation treatments could be selected for further study by finding those treatments that satisfy the above criteria.

These possible options could then be looked at in detail, with economic comparisons and other design and construction considerations being especially important.

Economic Comparison of Rehabilitation Options

This section will only provide an introduction to the costs that should be considered when comparing possible rehabilitation options. Further details are contained in the *Pavement Rehabilitation Manual* (Queensland Transport, 1992), and other publications relating to cost benefit analysis.

It is important that all pavements be analysed for the same period of time, to ensure a proper cost comparison. The costs calculated should be converted to present worth of costs, to take account of difference between future and present costs.

Construction Costs

These may be split into several categories. Material costs are often assessed using experience, and account should be made of possible variations in quantity required for various treatments.

Overhead and non-productive costs may include provision for traffic control, the cost of time lost to wet weather, and the need to dry and rework material, establishment costs, supervision and overhead costs, costs for relocation of services, and the costs due to material testing requirements.

Maintenance Costs

These are dependent on the pavement type. Granular pavements require routine maintenance from early in life, with periodic reseals or surface enrichments. Asphalt pavements require less routine maintenance than granular pavements, but the costs of a maintenance treatment are often greater. In addition, periodic resurfacing is needed. Cemented layers may require increased routine maintenance to seal reflective cracking.

Salvage Value

Factors influencing the salvage value of the pavement at the end of the analysis period may include the continued use of existing alignment, the feasibility of strengthening or upgrading the pavement using an overlay, the possibility of recycling existing pavement materials or asphalt in-situ or at plant, and the need to remove the pavement before reconstruction.

Road User Costs

These are difficult to quantify, but may include running user costs (vehicle costs) and user delay costs. Road user costs may be not included in the analysis if the options are similar in this regard, or if the road is minor. However, on heavily trafficked roads, the costs may be considerable and should be included.

Other Factors to consider when selecting a Rehabilitation Treatment

The following factors relating to design and construction should be considered, in addition to simply the costs of rehabilitation.

Effect on Public

The effect of the work on the public should be looked at. In particular, nuisance effects (dust, noise, smell), restriction of access during work, and environmental considerations (removal of trees for sidetracks, soil erosion, production of air pollution) may be important. Also, political sensitivities, the mood of road users, and the tolerance of any inconvenience should be examined.

Grade line Restrictions and influence on other Road Geometry

More costly treatments than simple overlays may be required to ensure that there is no rise in grade due to kerb or afflux considerations. Such options may include milling off and replacing existing top layer, in-situ stabilisation of existing materials, full reconstruction, or modified asphalt overlays.

The effect of the work on the present road geometry should be considered. Particularly important points to consider are horizontal and vertical alignments, and the cross-sectional shape.

Stage Construction

Stage construction may be adopted for some rehabilitation work. This may be done due to funding considerations, such as the economic optimisation of alternative rehabilitation strategies, or the need to apportion limited funds between equally important jobs.

It may also be done due to programming or scheduling constraints, such as the impending availability of an alternative route or reduction in traffic, the need to wait for a number of similar jobs to justify establish of special plant, and for the efficient allocation of available resources.

Traffic Management

Possible options include the detour of traffic for the duration of the project, side tracking only when work is in progress, carrying out work under traffic, or closing the site to traffic.

Factors to consider include the inconvenience to road users and the constructor, public safety, both vehicular and pedestrian, the need for provision of a safe working environment, and the cost of traffic provisions.

Traffic running on the work may have both beneficial and detrimental effects on construction. For example, while traffic loadings may identify weak spots prior to sealing, it may also cause ravelling or scouring of unsealed surfaces, and the smearing of road grime on interim surfaces.

Risk, Design Sensitivity, Construction Tolerances and Degree of Control

Factors difficult to control that affect the quality and performance of finished work include any variability of existing strength and materials characteristics, layer thicknesses, especially for in-situ treatments, locations of joints and transitions between different treatments

Operator skills with special plant and equipment, the variability of application rates and mix inconsistencies, and the sensitivity of the materials to environmental factors such as rainfall and temperature should also be considered.

Availability of Plant and Material

Factors to take into account may include whether purchase or hire plant should be used, plant establishment costs relative to the job size and the proximity of similar jobs that may use the same plant.

The scheduling of plant usage with the work program, sources of plant availability, and the availability of special materials may also need to be investigated.

3.9.3 Common Rehabilitation Options

This section lists rehabilitation treatments for granular and asphalt pavements that are commonly used, with the options for each failure type being listed approximately in order ranging from those suitable for minor failures to those suitable for major failures. More details about these options are described in the following sections.

Pavement Deformation

In minor cases, a thin asphalt surfacing or bitumen reseal may be sufficient. It is also possible to tyne, trim, compact and reseal the existing material, or provide a cold overlay (slurry seal). If there is evidence that poor drainage is the cause, the sub-surface moisture control should be improved.

For more severe cases, it may be necessary to carry out in-situ stabilisation, an overlay or reconstruction of the pavement.

Surface Texture Deficiencies

For bleeding and flushing, it may be possible to soften the binder and reapply the aggregate. An overlay with open graded asphalt can help to absorb the excess bitumen. A cold overlay (slurry seal) or bitumen reseal with adjusted binder rate may also be used.

In-situ stabilisation may be used if the cause is a weak base. The surfacing can be milled off and replaced with asphalt. In extreme cases, burning off the excess bitumen or hot-in-place recycling may be used. High pressure water retexturing is a new option that may be used.

If there is minor cracking, the cracks can be sealed using a crack-sealing agent. A polymer modified reseal can be used to prevent the risk of the surface cracking in the future. The use of a strain alleviating membrane interlayer (SAMI) and an asphalt overlay may be required. In extreme cases of cracking, it may be necessary to mill off or remove the surface, and resurface.

If poor drainage is thought to be a factor, it will be necessary to improve the surface or sub-surface moisture control. If it is occurring due to reactive soils and volume change within these, in-situ stabilisation may be useful. If the cracking progresses to a severe level, an overlay or pavement reconstruction may be required, or hot-in-place recycling may be used for asphalt pavements.

In the case of delamination, a bitumen reseal or asphalt overlay can be used. In severe cases, the defective surfacing should first be removed. To fix a loss of loss of skid resistance or polishing, a bitumen reseal, thin asphalt surfacing or cold overlay (slurry seal) can help improve skid resistance. In asphalt, it may be possible to lightly mill the surface. For isolated sections, patching could be used.

For excessive patching, a cold overlay (slurry seal) or thin asphalt surfacing may be used. It may also be possible to lightly mill the surface and bitumen seal.

For potholing, a bitumen reseal, cold overlay (slurry seal) or thin asphalt resurfacing may be used if the problem is with the surfacing. If it is due to poor drainage, the sub-surface moisture control or surface drainage should be improved. If the problem is due to a weak pavement, in-situ stabilisation, an overlay or reconstruction may be required.

For ravelling, if poor surface drainage is thought to be the cause, this should be improved. Rejuvenation or surface enrichment (a fog seal) may be used. For more severe cases, a bitumen reseal, cold overlay (slurry seal) or thin asphalt surfacing may be required. In severe cases, hot-in-place recycling may be used.

In stripping, a bitumen reseal, thin asphalt surfacing or cold overlay (slurry seal) may be used. Surface enrichment (a fog seal) or rejuvenation may also be used. It may also be possible to heat and replace the aggregate that has stripped.

3.9.4 Rehabilitation of Moisture Control and Drainage Systems

This section details the most common shortcomings that occur in a road drainage system, along with possible solutions. Many solutions may be feasible, and so the specific road cross-section details should be considered, since some treatments cannot be applied to all road types.

Problems on surface of road

Sheet flow is where there is excessive water on the road surface leading to spray and splash, and aquaplaning. If it is occurring, it will be necessary to increase the pavement crossfall, or improve surface water drainage.

If permeable shoulders and medians are a problem, it may be necessary to pave the shoulders and medians, taking care to ensure this does not block existing subsurface drainage.

Water trapped on an open graded base may occur, in which case it will be necessary to seal the surface, and any cracks, joints and discontinuities in it to prevent moisture entry. To help remove moisture it may be necessary to construct joint drains or longitudinal drains with lateral drains through the shoulder to allow the water to escape.

Moisture infiltration through cracks, joints and other discontinuities can be prevented by some type of sealing treatment. This may be a bitumen reseal, crack sealing treatment, asphalt overlay, or other treatment as required.

Moisture infiltration through permeable surfacescan be reduced by resealing the surface and/or providing transverse drains at location. It is also important to note that moisture may also accumulate at changes of pavement type, thickness or patches on longitudinal grades.

Problems at side of road

If the problem is shallow and/or silting table drains, it will be necessary to deepen the drain and/or remove debris from it. It may also be necessary to construct a kerb, gutter, gully pit and culvert system. In expansive embankments shallow table drains may be needed, in which case it may be necessary to provide flat batters to shift the water away.

Water seepage into pavement adjacent to median can be prevented by sealing the joints in the median slab tops and ensuring that the bedding sand is drained using weepholes. Strip drains constructed on the median edges can be used to help drain the pavement.

Moisture infiltration from a cutting can be reduced by placing a deep table drain or longitudinal subsoil drain under the shoulder. Transverse subsoil drains under the road at cut-fill transitions and permeability reversals are also desirable.

Problems due to construction practices

A reduction of the drainage capacity of kerbed roads may occur, especially due to overlays. It will be necessary to raise kerbs in conjunction, or pave the shoulders to allow water flow along these.

Blockage of the subsurface drainage may occur due to widening. However, normally proper design and inspection should prevent this. The widened section's base should have a higher permeability than the existing material. It may be necessary to construct edge lateral drains to stop water accumulating in the existing base.

Impermeable shoulders (boxed construction) can prevent water from leaving the pavement. A strip drain at the interface of shoulder and pavement can remove moisture, but it is important to ensure that no other drainage problems are introduced.

Problems due to other sources

Water ponding may occur due to incorrect road crossfalls, or against the edge of the pavement. A levelling course or patching work may be used for shape corrections. A slurry seal may be used for minor rut correction. The shoulder should be graded to the required crossfall and sealed if possible. If there is an accumulation of moisture in sags on vertical curves, it is necessary to ensure that water can drain sideways off and away from pavement.

Impermeable aggregate drainage layers may mean it will be necessary to provide an improved means of surface water interception and removal. It may also be necessary to provide a chance for the moisture to move out of the base material by the use of longitudinal collector drain systems.

Pockets of unstable subgrade may require that localised drainage systems are placed nearby to help. Replacement or in place stabilisation is only solution if the cause is poor material quality. Broken and clogged pipes and pipe outlets may also be a cause of this problem, and so the replacement or repair of observable damaged pipes and outlets, or back flushing of drainpipe and collector systems may be required.

3.9.5 Surface Treatments

For further details about these treatments, reference should be made to the *Pavement Rehabilitation Manual* (Queensland Transport, 1992), or other more up-to-date publications as relevant.

Sealing and Resealing

Sealing is application of thin layer of bituminous binder followed by aggregate. Resealing is the application of this to an existing surface. Aggregate size should be selected according to road characteristics.

The function of a seal is to protect the underlying pavement from water entry and other effects, and provide an abrasion and skid resistant wearing surface. A reseal normally aims to restore these above functions to an acceptable level.

Sealing or resealing should not be used in heavily trafficked tightly curved areas or heavy turning areas (unless polymer modified binder is used). It is also not appropriate for areas where traffic noise is problem. It is also useless for the correction of failures due to lack of structural strength (unless the seal is very thick), or the correction of deformation failures, unless combined with other rehabilitation treatments.

Dust Laying

This is the application of a low-viscosity, slow curing oil or bitumen to a dusty road surface. The function of dust laying is to avoid loss of natural binding materials from pavement as dust (when a reseal cannot be done immediately), and reduce the hazards associated with dust (limited visibility etc). Dust laying should not be used in conditions where no quick drying is to be expected (as water may lie on road, and reduced speed may be required).

Surface Enrichment

This is a light application of one of several types of bitumen (cutback, fluxed, emulsion) to an existing sealed surface, with no new aggregate being applied. It is a useful way of extending the life of the road, being much cheaper than a reseal. Surface enrichment may be used to enrich seals in which binder has oxidised, or initial binder application is low, and seal fine cracks of less than 1 mm width.

Surface enrichment should not be used in areas where traffic must travel on work soon after (e.g. busy roads), in low temperatures (less than about 20 degrees), or wet conditions (although the surface may slightly damp).

Rejuvenation

This treatment aims to restore an aged and oxidised pavement to a new and durable condition. The effectiveness is dependent on penetration of the agent and its interaction with the pavement bitumen.

Rejuvenation is not useful for the prevention of reflective cracking, to improve skid resistance, reduce road noise, act as a crack sealant, or for use on busy roads without sand.

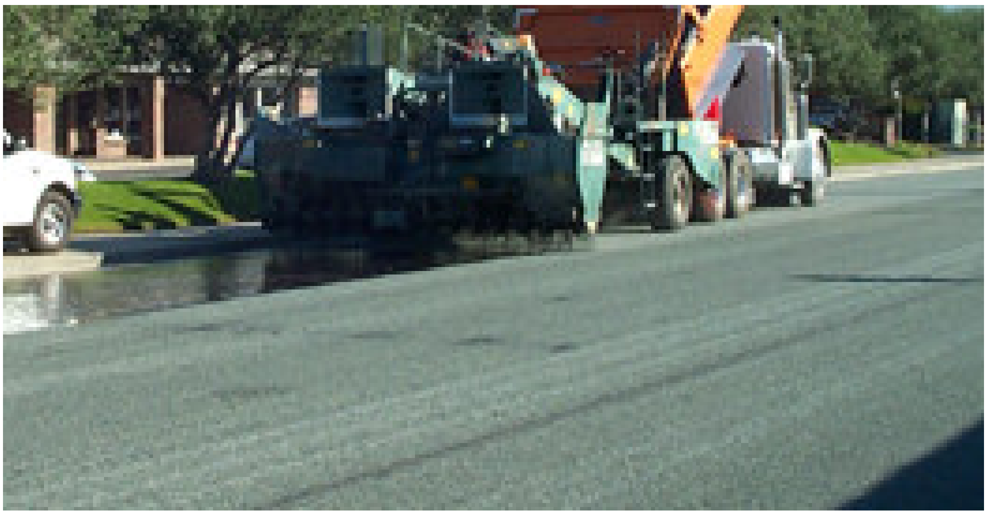


Figure 3.27 – Aggregate being placed onto top of freshly sprayed bitumen during a reseal

Figure 3.28 – Resealing:

a) Typically done in spreads about a lane wide

b) Rolling of the surface after the aggregate has been applied is important to ensure that the aggregate obtains a good bond with the binder

Joint and Crack Sealing

This seals joints and cracks, using a proprietary or purposely made compound. This should reduce moisture entry into the pavement, and consequent loss of strength.

Joint and crack sealing should not be used on a dirty or wet surface (adhesion will be poor), for the correction of expansive cracking (due to ground movement or temperature), or the correction of cracking that is due to pavement failure.

Polymer Modified Binder (PMB)

Polymer modifying agents can be added to binder used for any application. Generally, they reduce temperature susceptibility (both high and low) and give improved tensile strength, although at greater cost than conventional binder.

It is not a question of whether polymer modified binder is superior to conventional binder, but whether it is at a reasonable cost. The whole of life costs of both should be considered when making a decision. Important benefits, compared to ordinary binder, are listed below. The exact properties depend on the type of polymer-modified binder selected.

For asphalt applications, benefits may include reduced deformation at high temperature, and accelerated stiffening of mixes, improved fatigue resistance, reduced flushing and reflective cracking, rutting and shoving, and increased pavement durability and loading capacity. For spray seal applications, benefits may include reduced stripping and increased flexibility, reduced cracking and bleeding, and increased toughness and softening point.

A Strain Alleviating Membrane (SAM) is a seal coat using a PMB, as either single or multiple applications, the usual purpose of which is to reduce the incidence eof cracking.

A Strain Alleviating Membrane Interlayer (SAMI) is a single seal placed between an existing pavement and an asphalt overlay, to prevent tensile strains in existing pavement being transferred to overlay.

A Highly Stressed Seal (HSS) is a seal with a PMB designed to reduce aggregate stripping in areas of high stress such as sharp curves or steep grades.

Polymer-modified binders should not be used in situations of no particular benefit, where ordinary binder can be used at less cost, where pavement layers have failed, in adverse sealing conditions (such as low temperatures, wet conditions, or dusty stone), or in very thin asphalt layers.

Geotextiles

The purpose of a bitumen impregnated geotextile or fabric seal is to provide a waterproof membrane, and delay or prevent cracking. An additional purpose is to prevent the pumping of fines associated with cracking, leading to improved preservation of the load carrying capacity of the pavement.

Geotextiles should not be used to placement over fatigue cracking, in superelevated curves and stopping zones (due to slippage, but may be done with care), or on roundabouts or tight curves (where large shear forces will be applied).

Geogrids

This is a plastic fabric with relatively large openings, intended for use as integrated reinforcement in soil, rock fill or pavements. The use of geogrid enables a better bond between pavement layers above and below the geogrid. This leads to an improvement in the fatigue life of the pavement, and reduces the rate of crack propagation. Rutting may also be reduced.

Geogrids should not be used where asphalt recycling may be used in the future, if other cheaper crack rehabilitation treatments may be used satisfactorily, or in a thin overlay (they are difficult to place).

Figure 3.29 – Geotextile:

a) Placed within the pavement structure prior to material placement

b) Being placed as part of a fabric seal

Open Graded Asphalt (OGA)

This is a high void, open graded asphalt mix that allows water to drain through the layer. It may improve the skid resistance and safety of the surface, particularly during wet weather, and provide a quieter running surface. It may also be used to fix a bleeding surface, where excess binder is absorbed by the voids.

It is important that consideration is given as to how the water will drain away, once it travels down through the open graded layer. Provision for water flow should be made in the shoulder area. The layer below should be impervious and fairly smooth.

Open graded asphalt should not be used for the improvement of structural capacity or waterproofing if it is currently inadequate, for the correction of structurally unsound pavements, in situations where there will be an overlay of the open graded layer by a structural layer on top, or if the current top layer of the pavement is not impervious.

Slurry Seals

This is a mixture of continuously graded fine aggregate, mineral filler, bitumen emulsion, additives and water. It is applied as a resurfacing treatment. This treatment is applied to roads with good structural capacity, but with surface defects to be corrected.

Slurry seals should not be used in situations where structural capacity is required, or reflective cracking must be prevented, for the correction of ruts deeper than 15 mm with one application (which leads to segregation of the mix and flushing), where emulsion is slow setting, or on rejuvenated or cut primer sealed surfaces.

Figure 3.30 – Open graded asphalt:

a) The surface

b) A core, illustrating the open nature and high voids of this form of surfacing

Figure 3.31 – A view of a newly applied slurry surfacing (on the right), compared to the old surfacing

Hot In-Place Recycling

This is where the asphalt surfacing is heated, scarified, remixed, re-laid and rolled, in place. New material may be added to obtain desirable characteristics.

Hot in-place recycling should not be used if any of the lower pavement layers are not stable, excessive hardening of the binder has occurred, surfacing problems are due to problems with base or drainage, or there are large variations in surface thickness. It should also not be used if the pavement structure is weak, and cannot support mixing train and equipment, the pavement is very wet or the pavement contains geotextile fabrics.

Cold In-Place Recycling

This is where reclaimed asphalt is combined with new binder and/or additives to produce cold mix mixtures. New equipment may enable process to be done in a single pass. It may enable the structural capacity of the pavement to be restored, while conserving materials and energy.

Cold In-place recycling should not be used if any of the lower pavement layers are not stable, excessive hardening of the binder has occurred, surfacing problems are due to problems with base or drainage, or there are large variations in the surface thickness. It should also not be used if the pavement structure is weak, and cannot support mixing train and equipment, the pavement is very wet or the pavement contains geotextile fabrics.

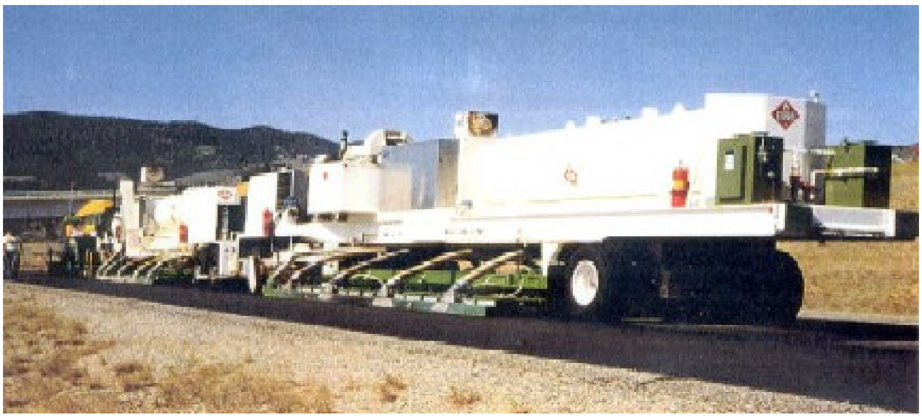


Figure 3.32 - A view of the equipment train used for hot-in-place asphalt recycling

Cold Milling

Cold milling removes a specified thickness of material mechanically, using a cutting bit to chip off the surface of the pavement. It may be used in conjunction with other techniques to restore the pavement to a newer condition, or improve the pavement cross-sectional shape.

Cold milling should not be used for the milling of a thin layer improve skid resistance, as roughness and ravelling may increase. Hence another treatment may be required in conjunction. Areas where there are manhole covers or geotextiles can cause considerable delays in work.

Figure 3.33 – Milling of a surfacing, with the material being deposited along the shoulder

3.9.6 Overlays

The purpose of overlays is to repair distressed pavements. Pavement failures that may require an overlay as part of the rehabilitation include severe ravelling or weathering, severe block cracking, structural cracking, excessive patching, rutting or shoving.

Asphalt Overlays

Asphalt overlays are used on pavements where seal coats should not be used, due to high traffic volumes. The purpose of the overlay is to add structural strength to the pavement, improve the skid resistance and driving quality of the pavement, and waterproof the surface.

An asphalt overlay is best where there is inadequate structural capacity of the pavement for current or future traffic loading, there is excessive noise generation, or unacceptable road user and maintenance costs are occurring. It is also ideal if there is high roughness, an unacceptable level of safety (skid resistance), poor riding quality or deep rutting, with water ponding in ruts and interfering with surface runoff

An asphalt overlay should not be used as the sole treatment for reflective cracking (it should be combined with other treatments), or as the sole treatment if there are level restrictions (it should be combined with milling). It also should not be used if the pavement layers are unstable due to poor material or drainage, or as the sole means of sealing the pavement against moisture entry (asphalt overlays are generally porous).

Unbound Granular Overlays

Granular overlays are a common form of pavement rehabilitation where traffic volume is less than about 2500 vehicles/day. The purpose of the overlay is to add structural strength to the pavement and improve the riding quality of the pavement.

A granular overlay is best if the cross section must be designed to provide outlets for various flow paths within pavement (extra drainage may be required). However, consideration must be given to relative permeability of overlay material, compared to existing material (to avoid trapping excess moisture in base).

Figure 3.34 – Two views of asphalt overlays being carried out, showing the placement machines and rollers

It is often used in composite treatments such as in-situ stabilisation of base and granular overlay, widening and overlay, or bitumen stabilisation and overlay.

A granular overlay should not be used if the pavement is saturated resulting from poor drainage or increase in water infiltration, if the pavement has faulty gradation of aggregate or lack of proper compaction (weakened especially when wet), or there is a weakness or failure of the subgrade.

3.9.7 In-Situ Stabilisation

There are many types of in-situ stabilisation that may be carried out, but the aim of all is to improve the properties of the material being stabilised, often to reduce volume change and plasticity, or improve strength and durability.

Mechanical Stabilisation

This is the improvement of existing pavement material by blending with one or more other types of granular materials. The effect may be to change the particle size distribution or the plasticity. This may improve the strength, compactability, impermeability and abrasive resistance of the pavement material.

Mechanical stabilisation should not be used if there are no satisfactory materials available, and cement or lime stabilisation is cheaper, or if the combined material mixture does not have the required properties.

Cement Stabilisation

This improves the properties of the existing pavement material by the addition of cement. This process is normally done in-place, and time must be given for the cement to cure. The pavement may be considered as cement modified, or cement bound, depending on the amount of cement added.

Cement Stabilisation should not be used to fix problems occurring lower down in the road formation, or in areas with many utilities, as these will hinder or stop the process. If the subgrade is saturated due to moisture entry, it should be restored prior to treatment. If constant depth mixing is used, some subgrade material may be mixed in with the pavement material, and this should be taken into account.

Lime Stabilisation

This improves the properties of the pavement material, by the addition of lime. It is generally similar to cement stabilisation. However, the rate of strength gain is much slower, and it is more suited to heavy clays, rather than granular materials. Lime stabilisation may be used to improve inferior pavement material, or to improve the subgrade, which is usually clay.

Lime should not be used if rapid strength gain is required, or if there are no clays or other pozzolanic material within soil to react with the lime. It should also be noted that low temperatures and high organic contents slows the reaction, and cohesionless materials may require secondary additives.

Bituminous Stabilisation

Bitumen stabilisation may be done using bitumen emulsion, cutback bitumen or penetration grade bitumen (foamed bitumen). Foamed bitumen enables the coating of fine-grained soil particles, due to low viscosity and necessary surface tension. Some bitumen contains anti-foaming agents that may hinder the process, unless they are neutralised. Bitumen stabilisation aims to give some cohesion to non-plastic materials, or to make a cohesive soil less prone to loss of strength and stability with increasing moisture.

Bitumen stabilisation should not be used if the soil is not capable of being pulverised, and is not suitable for fine-grained soils unless pre-treated. Stabilisation with bitumen that requires aeration before compaction should not be carried out if rain is likely to occur before completion.

Figure 3.35 – A series of pictures showing the in-situ stabilisation process: before treatment, after stabilisation agent has been placed, mixing of stabilisation agent into soil, after mixing and ready for resealing.

Other Stabilising Agents

This includes stabilisers other than lime, bitumen and cement, of which the main types are chlorides, miscellaneous chemicals and proprietary products. It is important that the properties of the stabilisers be carefully examined and tested, to ensure the best and cheapest stabilising agent is selected that suits the particular application.

3.9.8 Miscellaneous Rehabilitation Treatments

This includes treatments that are relatively new or that do not easily fit into the previous categories. However, they are valid options if used where there is evidence that their use will be both successful and cost effective.

Fibre Reinforced Seals

This uses a polymer-modified emulsion with glass fibres acting to reinforce the binder. They are a possible alternative to geotextiles, or other PMB treatments.

The general process involves spraying an emulsion tack coat over the surface, followed by application of chopped fibreglass strands. This is then followed by a second application of emulsion, and aggregate spread in the usual way.

Stone Mastic Asphalt (SMA)

This is a gap graded asphalt mix that contains a large proportion of coarse aggregate. This interlocks to form a strong structure with resistance to rutting and shoving.

A high binder and filler content binds the aggregate particles together and fibres are normally added to improve stability and drainage. This gives a surfacing with fatigue resistance and a long life, normally being used as a thin surfacing.

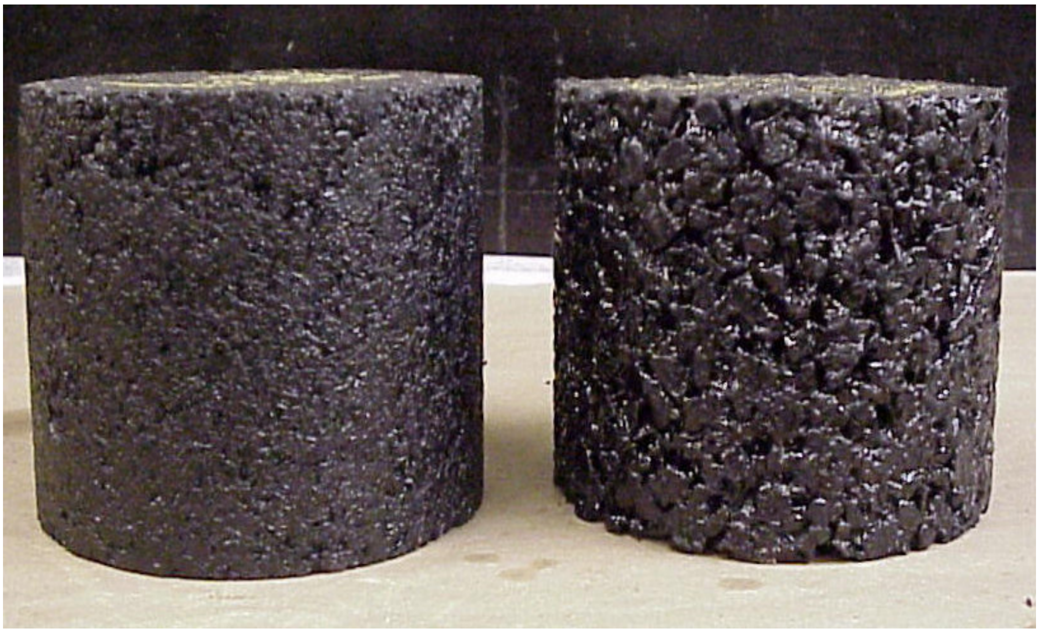


Figure 3.36 – A comparison of cores from dense graded asphalt (on left) and stone mastic asphalt (on right)

Figure 3.37 – A view of a Stone Mastic Asphalt surfacing, illustrating the high proportion of large particles present in the structure

Novachip

A proprietary process, it consists of hot mixed aggregate of 10-14 mm size with fines, but fairly open graded. It is laid onto a continuously applied emulsion spray, all being done with one piece of equipment.

While thin, it can correct deeper depressions. Its stone to stone contact gives stability and resistance to deformation is good. It has the water shedding and noise reducing properties of an open graded mix, while the emulsion seal makes the layer quite impermeable.

High Pressure Water Retexturing

This method uses water applied at high pressure to remove excess binder from the surface of the road. Trials have shown it is successful in increasing both surface texture and skid resistance. Best production rates are generally achieved in cooler weather.

While the cost of treatment is approximately double that of resealing, a reseal does not remove excess binder, which is the cause of the problem, and so failure may reoccur.

This method is not recommended for pavements with only initial primer or primer seals since the high-pressure water may reduce the integrity of the seal and expose the underlying pavement layers to moisture entry.

Other

Other rehabilitation treatments may also be used, based on local knowledge and experience. However, unless the treatment is being using as a trial for the first time, it should only be used if there is good evidence from previous treatments that it is likely to be effective in the intended application.

Figure 3.38 – A picture showing a surface before water retexturing (on right) and after water retexturing (on left), showing the large increase in surface texture

Figure 3.39 – High pressure water retexturing:

a) A view of the mechanism used

b) A view of a typical vehicle

3.10 Report on Outcome of Investigation

3.10.1 Introduction

Once the cause of a pavement failure has been determined, there is a need to try and prevent a reoccurrence in the future. To help achieve this aim, a forensic investigation report should be prepared.

This report should simply detail all information gathered during the investigation in a logical and systematic way, so that others reading it can easily learn from the findings. Information that should be included is discussed below.

3.10.2 General Review of Project and Location

This should provide information about the history of the road section, a brief description of the project on which the failure occurred (if the failure is project-specific), and any other relevant details concerning the location.

3.10.3 Failure Details

This should provide information about the failure that is occurring, and should include information about the type(s) of failure and its extent, and any other relevant details.

3.10.4 Testing carried out

This should provide information about any testing that was carried out, and the results. It should include information about both the non-destructive testing, such as visual inspection and deflection testing, and the destructive testing, including trenching, standard tests and other tests used.

3.10.5 Probable Cause(s) of Failure

This should detail what the probable cause(s) was found to be, along with any evidence to support this conclusion. Any uncertainties regarding this failure cause(s) should be included.

In addition, any recommendations to prevent a reoccurrence of this type of failure in the future should be listed.

3.10.6 Rehabilitation Options

This should provide information about the best rehabilitation option for the failed pavement, and the steps in the process to arrive at this conclusion.

Approximate costing of the desired option and other possible options should be included, for comparison.

3.11 Identification of Pavement Failures at the Network Level

3.11.1 Introduction

This section is different to the investigation method and case studies, where specific pavement failures were examined. In this section, a simple methodology for determining the location and possible causes of pavement failures at the network level will be developed.

This methodology is related to the investigation methodology previously developed, and may be a preceding step, since it is not possible to investigate a pavement failure, unless it is known to exist.

While pavement failures are often revealed by inspection of the road or complaints from road users, there is typically some bias so to how bad the failure really is. This methodology aims to give an impartial method for the discovery and examination of possible pavement failures at the network level.

The methodology developed consists of the following steps, which will be discussed in more detail below:

- Obtain road condition data
- Set pavement failure limits, and find sections that exceed this
- Analyse selected sections for a probable cause of failure

The methodology will be discussed with reference to the Queensland Department of Main Roads. While the exact software and information available will vary in other road agencies, the broad principles are similar.

3.11.2 Obtain Road Condition Data

In the Queensland Department of Main Roads, all road condition data is stored in the ARMIS (A Road Management Information System) database. The program Chartview allows easy selection and viewing of required data. The general procedure for the gathering of road condition data from Chartview is discussed below.

Firstly, the required road section must be selected. This would normally be a road within the local district. Next, the particular carriageway number is chosen. The terminology used within the Queensland Department of Main Roads is as follows:

1 = Lanes on a two-lane two-way road (undivided carriageway)
2 = In gazettal direction on a divided carriageway
3 = Against gazettal direction on a divided carriageway

In Southern district, the majority of roads only contain a number 1 carriageway. The notable exception is Warrego Highway, 18A (Ipswich-Toowoomba), which is divided carriageway for most of its length. Roads may often contain varying carriageway numbers for different sections, and this should be taken into account, when collecting the road condition data.

Next the desired road condition data should be collected, most easily done by copying the data into a spreadsheet. Possible information that may be used for assessment of pavement failures may include the following that is discussed below.

- Jobs details
- Pavement layer information
- Traffic details
- Roughness
- Rutting
- Surface Texture

Job details gives details about all jobs carried out on the selected carriageway of the road section. It is useful for giving an idea of the recent construction history along a road section.

Pavement layer information contains information on the age, types and thicknesses of pavement layers contained at certain chainages along the road. It can be important information.

Traffic details may include the AADT and more importantly the number of commercial vehicles per day. It is used as an indication of the importance of the road section, and possible failure causes.

Roughness provides a good indication of the general performance and drivability of the road and likely perception by road users.

Rutting provides an indication of any structural failure or deformation of the road pavement that may be occurring.

Surface texture is normally measured at the network level using laser profilers. It can be used to give an indication of skid resistance levels (which are tested less frequently), and also to assess the amount of bleeding or flushing that is occurring.

Apart from the above information, other data may also be useful e.g. that relating to cracking, and could be used as required. The data may be collected at varying intervals, normally at 100 m or 1 km intervals.

3.11.3 Determine Failed Pavement Sections

Once the pavement condition data has been obtained, there must be some process by which the pavement failure limits are set, and sections that exceed this are found. Varying conditions may be used separately or as a combination e.g. roughness, rutting, surface texture, or a combination of these.

Failure is a subjective term and in this methodology is considered to consist of pavement sections that exceed the specified limits. However, these sections are normally still suitable for use by road users. The failure is often not quick and spectacular, but rather a slow and steady form of failure.

The failure limits can be set as either relative or absolute. An example of a relative limit is to define a failure as roughness exceeding the 90% roughness value for the road. An example of an absolute limit is to define a failure as roughness exceeding a value of 120.

Relative and Absolute Pavement Failure Limits

Using relative limits allows the acceptable roughness and rutting to be based on the existing condition of the road. To improve the overall condition of the road, it makes sense to improve the worst sections first (depending on other factors, such as traffic or geography), and these can be found by using relative limits.

One relative measure is the percentile value, which is calculated by ranking the data values in descending order, and dividing the rank by the total number of values, and expressing this as a percentage. This is best done in a spreadsheet, and could be done using the data from multiple road sections. Another relative measure is to use a failure limit that is a percentage of the above value. For example, failure may be defined as sections where the roughness exceeds 150% of the average over the road section.

Using absolute limits means that the failure limits must be arbitrarily set, although they may vary for differing road classes. This may be an advantage in the prioritisation of road rehabilitation on multiple roads, where an absolute measure is required. However, for the identification of pavement failures on a particular road, there are no advantages over relative limits.

Whether the limits are relative or absolute the examination of the data for sections that exceed this can be done in two ways. The graphical method is simplest, while the numerical method provides more precise information, although taking longer.

Graphical and Numerical Examination of the Data

Graphs can be plotted showing the roughness and rutting variation along the road. Then, acceptable failure limits for roughness and rutting can be plotted, highlighting any sections above the limits. Limits may be absolute or relative, and set by calculating certain percentile values e.g. 90% roughness. From a visual examination of the graphs, sections where the failure limits are exceeded can easily be seen.

Examining the data using a numerical method is generally similar to the graphical method in that it compares the data values to set limits that may be either relative or absolute. In contrast to the graphical method however, the comparison of the data to the limits is done numerically, most conveniently in a spreadsheet. This method may actually be quicker than visual observation of the graphs, although these should always be checked for confirmation of the results.

3.11.4 Analyse Failed Pavement Sections

The analysis of the selected pavement failure sections for a probable cause of failure can only be done to a limited extent, based on the information available. For example, there is no way to tell if moisture or soil properties are contributing factors. However, useful results can still sometimes be found.

To determine the possible cause(s) of failure, the seal age, pavement age, pavement depth and traffic volumes should be examined. For example, if there is high roughness and rutting, and the seal is old, then this may be the primary cause. High traffic volumes, or an old and thin pavement may also be contributing factors.

Chapter 4

Investigation of Pavement Failures

4.1 Bowenville-Dalby

4.1.1 Plan the Investigation

General Review of the Problem

The 67/18B/302.1 project was the first of several sections of widening and overlay planned for the Bowenville-Dalby section of the Warrego Highway, between Toowoomba and Dalby (18B). The project was located between chainages 74.7 and 76.2 km, a length of 1.5 km, and involved the widening and overlay of the existing 9m wide pavement to a fully sealed 11m formation.

The job was constructed between September and December 2002 by RoadTek, who received the open tender contract. Further details about the construction process will be discussed later, and they are important, having been a factor contributing to the failure.

In June-July 2003 (about 7 months after the completion of the construction), and after a period of constant steady rain, the project showed signs of premature failure. This consisted of bleeding/fatty depressions and shoving along the edgelines of the traffic lanes. These early failures continued to deteriorate, and necessitated the rehabilitation of outer widths of the entire section in June 2004.

Figure 4.1 – Bowenville-Dalby:

a) A view of the excess binder along the outer edgeline and outer wheelpath

b) A closer view of the fatty surface near the edgeline

Scope of Work

Importance of the road

The Warrego Highway between Toowoomba and Dalby (18B) is part of the National Highway system and an important road link for the Darling Downs. As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

The traffic volumes on this road are high for a rural road. In 2003, the AADT (Average Annual Daily Traffic) in the area of the job was about 3650 vehicles/day, including about 680 heavy vehicles, composing about 18% of the total traffic volumes. These heavy vehicles include a significant number of Type 1 Road Trains and B-Doubles, in addition to Semi-Trailers. Hence, the use of the road for the transport of freight is very important.

Magnitude of the failure

In addition to excess binder on the road surface in the outer wheelpaths, deformation also began to occur in the outer wheelpaths. A surface profile measurement was undertaken in October 2003 on the westbound lane at Ch. 75.55. This revealed that the deformation in the outer wheelpath was occurring to a depth of about 20 mm below the required level. However, this material was being pushed up to the outside, meaning the effective deformation was about 30-40mm.

This indicated that the failure occurring was beginning to reach unacceptable levels, since this deformation prevented water from flowing off the pavement during rainfall, leading to further deterioration.

During an inspection in October 2003, the length of road affected by the failures was recorded, as shown in the table.

As can be seen, the length of the job affected by the failure was almost 25%, even in the early stages. As time went on, the length affected both westbound and eastbound continued to increase, until almost all the length of the job was affected, with the failures being concentrated in the outer wheelpaths.

Table 4.1 – Length of failures on Bowenville-Dalby 67/18B/302.1 during October 2003 inspection

Location	Westbound	Eastbound
No. of Failure Sites	17	25
Failure length (m)	339	271
% of length of Job	22	18

Current risk to road users and risk of deterioration in the future

In the early stages, the failure posed little risk to road users. However, as the deterioration increased so did the risk, although generally the road was still quite safe. When first noticed in June-July 2003, the failures were minor. Hence, it was decided to see how the deterioration progressed over time. The failure continued to increase in both length and magnitude, until in January 2004 almost the whole length both ways was affected in the outer wheelpaths. This meant that rehabilitation work was necessary.

Planned future work

Originally a second seal was planned for December 2003, using carryover funding from the original job. This was not carried out, due to the failures occurring. In June 2004, the section was rehabilitated.

Investigation Plan and Team

The goal of the original investigation, commenced in October 2003, was to determine the cause of the premature failure and how to rectify the failures so that future planned works did not fail due to the same problems.

The investigation aimed to carry out the investigation in a cost-efficient manner, to ensure that while useful information was obtained, it was not at too great a cost.

A visual investigation was first conducted to observe the failure and determine what testing might be necessary to carry out. Next, the materials sampling and testing was carried out, to help formulate the failure cause(s) and the best rehabilitation treatment.

The investigation team for the visual investigation carried out in October 2003 consisted of the two Maintenance Engineers, and the Area Inspector and Engineer.

4.1.2 Review Documents and Literature

Plans

The plans for the job and a typical cross-section of the work were reviewed. This provided information about the work to be carried out and other information. The design life of the project was 10 years, with the design traffic loading being 3.8×10^5 ESA's. The assumed su bgrade CBR for design was 5 and the nominal road crossfall 3%.

Hollows in the existing 9m wide seal were filled with a 10 mm asphalt corrector course to improve drainage and fix the profile of the existing surface. The existing shoulders were then boxed out 200m deep and widened with Type 3.4 material.

The widened shoulders were primer sealed with AMC4 cutback bitumen and 10 mm aggregate. A C170 fabric seal with 10 mm aggregate was placed on the widening overlapping onto the existing sealed pavement. This fabric seal was placed to reduce any cracking propagating to the surface at the join between the existing and widened sections.

The existing pavement was shape corrected to 300 mm below the final grade level using Type 3.1 gravel, and the granular overlay material, 300 mm of type 3 gravel, was placed on top in two layers.

A primer seal of ACM4 cutback bitumen and 10 mm aggregate was sprayed, and the final seal, a polymer modified binder S2S seal with 14 mm aggregate was placed on top.

This basic construction process was slightly modified due to traffic requirements, as discussed below. In October 2002, the decision was made to increase the basic road crossfall from 3% to 4% for the shoulders only (outside the road edgeline). This was done to reduce the possibility of problems due to the reactive black soil.

Pavement History

From the Main Roads asset management program ARMIS, the following information was found, regarding the pavement history over the length of the job (chainage 74.7 – 76.2 km) prior to the recent overlay

As can be seen, the job history of the section prior to the granular overlay and widening was relatively uncomplicated, with the initial work in 1976 being followed by two reseals at 9-year intervals.

Table 4.2 – Pavement history on Bowenville-Dalby 67/18B/302.1 prior to recent overlay

Date	Type	Description	Chainage (km)
May 1976	Initial Seal	Natural Soil Subgrade 200 mm Granular Material Spray Seal	74.6 – 78.5
April 1985	Reseal	Spray Seal	74.5 – 78.4
April 1994	Reseal	Spray Seal	74.6 – 78.5

Pavement Materials Information

Type of material(s)

A 10 mm asphalt corrector course was used for profile correction of the existing surface, and type 3.4 gravel was used for boxing out the existing shoulders for the widening of the road. The widened shoulders were primer sealed with AMC4 cutback bitumen and 10 mm aggregate. The design spray rate was 1.2 L / m^2 and the 10 mm aggregate design application rate was 1 m^3 / 130 m^2.

A C170 fabric seal with 10 mm aggregate was placed over a width of 4m, overlapping the join between the widening and the existing seal. The PF1 fabric was 4m wide, the C170 design spray rate 2.2 L / m^2 and the 10 mm aggregate design application rate was 1 m^3 / 130 m^2.

Type 3 gravel was used for shape correction and the overlay, which consisted of a depth of 300 mm placed in two separate layers. This material was sourced from the Wagners Quarry at Malu.

The primer seal consisted of ACM4 cutback bitumen with 5% cutter and 10 mm aggregate. The design spray rate was 1.2 L / m^2 and the 10 mm aggregate design application rate was 1 m^3 / 130 m^2. The final seal consisted of a polymer modified binder S2S seal with 14 mm aggregate. The design spray rate was 1.8 L / m^2 and the 14 mm aggregate design application rate was 1 m^3 / 100 m^2.

Required specification of material properties

The Queensland Department of Main Roads Standard Specifications were used for this project. The most relevant specifications included MRS 11.05 – Unbound Pavements, MRS 11.17 – Bitumen, MRS 11.18 – Polymer Modified Binder and MRS 11.22 – Supply of Cover Aggregate. In addition, the following relaxation of the specifications was allowed for the base gravel used for the overlay:

A modified 'C' grading was specified, with the minimum % passing 0. 425 mm and 0.075 mm sieves to be 17% and 9% respectively. Linear shrinkage was allowed to be between 3% and 5%. A plasticity index of between 6% and 8.5% was allowed, and a minimum CBR of 60, measured at 100%MDD and 100% OMC.

Quarry stockpile test results for the gravel

Samples taken for particle size distribution (grading) testing on 31/10/2002 and 11/11/2003 were within the limits as set in MRS11.05 for C grading, and satisfy or almost satisfy the other grading requirements above.

A sample taken for CBR testing on 11/11/2002 had a CBR value of 180, which was much greater than the value of 60 that was required. Other results from testing on the quarry material are compared to the specified values in the table below. Values in brackets refer to any relaxation of the specification values granted by Main Roads, as mentioned above.

Table 4.3 – Quarry stockpile test results for granular material used on 67/18B/302.1

Test	MRS11.05 – Type 3.1 (relaxation allowed)	31/10/02	11/11/02
Liquid Limit (%)	25 max	23.6	21.4
Plasticity Index (%)	6 max (6-8.5)	8.4	5.6
Linear Shrinkage (%)	3.5 max (3-5)	4.8	4
Plasticity Index x % < 0.425mm	150 max	141	81
Linear Shrinkage x % < 0.425mm	85 max	81	58
Ratio 0.075/0.425	0.35 – 0.55	0.54	0.56

As can be seen, the relaxed specifications are mostly satisfied by both samples. As can also be seen the later sample has improved material properties compared to the earlier sample, possibly indicating improvement of the material production by the Wagners Quarry from which the material was sourced. The latest sample, taken on 11/11/02 is taken to be representative of material prior to placement in the overlay.

Base test results prior to sealing

These test results include ball penetration tests carried out on the westbound pavement surface on the 18 – 22 /11/2002, prior to primer sealing. Five samples were taken on each day. This measurement can be used to adjust the aggregate spread rate to compensate for aggregate embedment. These tests are summarised below, and show that the completed surface was generally quite hard.

Other testing carried out on the granular material in the overlay included moisture content, degree of saturation and dry density testing, carried out from 6 – 22 November 2002.

This testing is summarised below, with only the average values shown for simplicity. When comparing the values to the specifications, characteristic values must be calculated, based on the number of samples and the standard deviation. The process by which this is done is outlined in MRS11.01 – Introduction to the Standard Specifications.

As specified in MRS11.05, for a type 3 unbound gravel the characteristic degree of saturation should be less than 70%. This is satisfied by all of the above, although the degree of saturation on the right side of the project was much greater than that on the left. The minimum characteristic dry density ratio for a type 3 unbound gravel is specified in MRS11.05 as 100%. This is satisfied by all of the above.

Table 4.4 – Ball penetration test results on 67/18B/302.1 base prior to sealing

Date	Range of Samples (km)	Mean Ball Penetration (mm)
18/11/02	74.7 – 75.1	2.7
22/11/02	75.2 – 76.1	3.6

Table 4.5 – Moisture, density and degree of saturation test results for 67/18B/302.1 base material

Side	Layer	No. of Tests	Moisture Content (%)	Dry Density Ratio (%)		Degree of Saturation (%)	
			Mean	Mean	Char.	Mean	Char.
WB	Lower	30	3.5	101.1	100.4	52.6	56.1
	Upper	30	3.4	101.1	100.4	50.6	54.6
EB	Lower	36	3.8	101.2	100.1	58.8	64.8
	Upper	33	4.0	101.6	100.8	62.9	68.0

Construction records

Testing methods and frequencies

As found from the tests above, there were about 130 tests carried out on the granular overlay material being placed. The total volume of the material in the overlay is approximately 0.3m x 11m x 1500m = 4950 m^3.

This corresponds to a testing frequency of about 1 test for every 38 m^3, or about 1 test every 11.5m along the job. This testing frequency seems about right for this type of project.

Other Information

Due to traffic requirements, the project had to be constructed in two parts. This enabled traffic to still use the road side on which no construction was currently being done. The northern (eastbound) side was overlayed and primer sealed first, before overlaying the southern (westbound) side. During this process the primer seal on the northern side failed. The seal was maintained until the southern side was complete.

The failed primer seal on the northern side was removed, and a new primer seal was sprayed full width. Importantly, while removing the primer seal from the northern side, the grader made gouges and tears in the surface, mostly along the outer wheelpath and edgeline at the location where crossfall changed from 3 to 4%. These areas were not restored to a hard and dense surface.

During construction of the southern side, the project was saturated with rain in one night, with most moisture penetration occurring from Ch. 75.18 – 76.2 km.

Water had also penetrated sideways into the northern base, which had been previously completed. This affected area was pulled out, dried and relayed. Moisture content testing indicated that this was effective.

The project was constructed under traffic control during the day, with the full road open to traffic each night, at a reduced speed limit.

The soil is known to be a reactive black soil. However, since the overlay was placed on top of the existing pavement and added shoulders, the effect of this could be expected to be minimal.

As mentioned above, rainfall occurred during construction, but the affected material was assumed to have been dried out sufficiently. Heavy rainfall occurring in June-July 2003 was a secondary cause of the initial failures that started to occur.

Published Articles

A preliminary report, *Investigation of Failures on Job 67/18B/302.1* (2004), was produced by the Infrastructure Delivery department of Southern District. This report listed the results of the visual investigation and materials testing, and hypothesised on the probable cause(s) of failure, and possible rehabilitation work. This report was the source of much of the information used for this investigation.

4.1.3 Interview Personnel

A maintenance engineer was of the opinion that the failures were due to a combination of factors that are discussed below.

The aggregate embedment on the northern side was due to gouges and tears made by the grader when tearing up the original primer seal on this side. There was a lack of surface densification on southern side, due to base being laid and primer sealed within a week.

There was embedment of the aggregate in the primer seal. This led to flushing occurring outside the normal outer wheelpath location, due to the lack of edgelines causing the traffic to track wider.

The final seal was sprayed using one rate in the wheelpaths and another rate outside these areas. These rates were sprayed to align with the final edgelines, but this did not take account of the flushed outer wheelpaths of the primer seal. This led to a heavy binder level being placed on top of another, and resulted in heavy flushing from 300 mm inside to 300 mm outside the edgelines.

The flushed up areas have had water forced through the surface by heavy vehicles, and has resulted in wetting of the base material under the seal. This has led to further aggregate embedment and plastic deformation of the upper part of the base gravel.

A materials officer was of the opinion that the failure was due to material breakdown. Normally, this would be unlikely to occur for the crushed rock material used for the overlay, but this hypothesis was further tested based on the materials samples taken, as will be discussed later.

4.1.4 Non-destructive Condition Survey

Visual Examination

A visual examination of the failures was conducted in October 2003, about 10 months after the end of construction, and about 4 months after the failures first began to occur.

The failures consisted of a flushed seal outside the outer wheelpaths along the edgeline and adjoining sealed shoulder, ranging from about 300 mm inside the edgeline to 300 mm outside it. .

The failure was found to be occurring over a total length of about 340m (average failure length 20m) on the southern side, and 271m (average failure length 11m) on the northern side. The failures occurred along the length of job.

It was observed that the surface was being laterally displaced, predominantly in the outer wheelpaths, causing ridging up along just outside the edgeline. Some stones under the original edgeline had moved sideways up to 60mm, indicating a softening of the upper base material, allowing plastic deformation. Some of the movement may have occurred in the seal.

Rutting was starting to occur in the outer wheelpaths, but not in the inner wheelpaths. The seal seemed to be intact, indicating that moisture could not be entering the pavement through cracks in the surface.

Other Forms of Non-Destructive Testing

The deformation in the early stages occurred predominantly in the outer wheelpaths. A surface profile measurement was undertaken in October 2003 on the westbound lane at Ch. 75.55 km. This revealed that the deformation in the outer wheelpath was occurring to a depth of about 20 mm below the required level. However, this material was being pushed up to the outside, meaning the effective deformation was about 30-40mm.

Figure 4.2 – Rutting:

a) Near the edgeline, with the material being displaced to the sides

b) A view of rutting and the excess binder near the surface

Figure 4.3 – Wavy edgeline:

a) A view along the length of the job

b) A closer view

4.1.5 Destructive Materials Sampling and Testing

Determining the need for Materials Sampling and Testing

For this failure, the following factors were reviewed when determining whether materials sampling and testing should have been carried out:

The road is a major transport link, and it would be desirable to ensure that the best rehabilitation treatment was selected. The failure was progressively increasing in both extent and magnitude, and so a strategy of doing nothing would probably not have been cost effective, as it would have necessitated the need for much more expensive reconstruction later.

The cause of the failures was not conclusively determined from a visual investigation of the pavement, and so it would be desirable to carry out materials sampling and testing to confirm or revise the failure hypotheses formed. The fact that this project was only the first of several similar projects in the same area gave an added need to determine the cause of the failure, allowing it to hopefully prevented in the future.

The cost of testing, when compared to the total cost of the investigation and the cost of possible rehabilitation treatments, was quite minimal and within acceptable levels. The possibility of further pavement deterioration if a poor rehabilitation treatment was selected meant that the money spent on the materials sampling and testing would probably be money well spent.

Trenching – Part 1

Two sites were selected for trenching of the pavement. One site was in a failed section, while the other was in a section currently experiencing no failure. This enabled a comparison to be made between the two sites.

Figure 4.4 – Trenching Part 1:

a) Trench being dug. There appeared to be little bond between the seal and base.

b) The base gravel was wetter under flushed areas where the failure was occurring

A mini excavator on rubber tracks was used to excavate the trenches on 21 October 2003, and in addition to materials samples taken for testing, a visual observation of the pavement structure was completed. It was observed that the seal peeled off very easily in both the good and failed sections, indicating little bonding to the pavement, and there was no penetration of the primer seal into the surface of the base gravel.

The base gravel was soft and very easy to dig, and this was more the case in the failed section. The moisture profile seemed to be consistent throughout the depth of the base layer, and the base appeared to be wetter in the failed section than the non-failed section. In both trenches it was seen that there was no accumulation of moisture at the interface between the two 150 mm layers placed in the overlay, and there was no separation of these layers. The depth of the top base layer at the trenched locations was measured as 140mm.

The testing carried out on samples taken from both trenches included moisture contents taken at various locations on the road profile, particle size distribution (grading) and Atterberg limits. The tests were carried out over a 150 mm depth, and so the results represent the average values in the base layer. The test results are discussed in the following section, with a summary being provided in Appendix E.

Moisture Content Test Results

The moisture content test results from the initial materials sampled are summarised in the table below.

Table 4.6 – Moisture content test results from initial materials sampled

Location	Distance from centreline (m)	Details	Moisture Content (%)
Failed	2.2	Between wheelpaths	3.8
	2.9	On inside of failures	4.6
	3.8	Just outside edgeline	4.3
Good	2.2	Between wheelpaths	3.1
	3.6	Near edgeline	3.3

From the table, it can be seen that the moisture contents in the failed trench were considerably greater than those in the trench in the non-failed section. The moisture content increase was greatest near the edgeline, where the failures were occurring, but the moisture content also increased further in, between the wheelpaths.

Also, it may have been be possible that the moisture content in the non-failed section was starting to increase near the wheelpaths, indicating that failure may start to occur here. However, due to the limited number of samples taken, this was not conclusive.

Particle Size Distribution (Grading) Test Results

These results were compared to the most recent original quarry stockpile test results, as well as the required specifications. The testing results from the trenching were those from the trench in the failed section.

Table 4.7 – Grading test results from initial materials sampled, compared to quarry results

		Quarry	Failed Trench: 21/10/03	
AS Sieve Size (mm)	Specified C Grading	11/11/02	2.2m from CL	3.8m from CL
37.5	100	100	100	100
19	80 – 100	99	98	99
9.5	55 – 90	64	64	64
4.75	40 – 70	47	47	49
2.36	30 – 55	31	31	34
0.425	12 – 30	14	17	18
0.075	5 – 20	8.1	11	11

From the above results, it can be seen that the both samples taken from the failed trench still satisfy the grading specifications. However, it can also be noted that the % of material passing the smaller sieve sizes (2.36 mm and smaller) increased in the trench samples, when compared to the original quarry results. This was especially true in the sample taken from 3.8m from the centreline, where the failure was concentrated.

From these test results, it appeared as if some breakdown of the gravel in the base was occurring, but the material still meets the specifications. With no testing having been done to assess whether the material was prone to breakdown, it is difficult to assess whether this was a contributing cause of the failure.

Other Test Results

These results were compared to the most recent original quarry stockpile test results, as well as the required specifications (and any relaxations allowed). The testing results from the trenching were those from the trench in the failed section.

Table 4.8 – Other test results from initial materials sampled, compared to quarry results

	Specifications	**Quarry**	**Failed Trench: 21/10/03**	
Test	**MRS11.05 – Type 3.1**	**11/11/02**	**2.2m from CL**	**3.8m from CL**
Liquid Limit (%)	25 max	21.4	24.2	22.8
Plasticity Index (%)	6 max (6-8.5)	5.6	8.4	6.2
Linear Shrinkage (%)	3.5 max (3-5)	4	5.6	4.6
Plasticity Index x % < 0.425mm	150 max	81	142	112
Linear Shrinkage x % < 0.425mm	85 max	58	95	83
Ratio: 0.075/0.425	0.35 – 0.55	0.56	0.67	0.61

From the above results, it can be seen that all of the properties had increased in both samples. The liquid limit, plasticity index and weighted plasticity values were still within the specifications. Linear shrinkage had one of the samples outside the relaxed specifications. Both weighted linear shrinkage values exceeded or nearly exceeded the specifications. The ratio of values passing the 0.075 and 0.425 mm sieves was outside the specifications.

Interestingly, the grading results suggested that breakdown was occurring more at 3.8m from the centreline, near the outer wheelpath, where the failures are occurring. From this, it would be expected that the greatest change of properties in the above table would have also occurred at this point.

However, from the above results, it can be seen that the greatest change in properties occurred in the sample taken at 2.2m from the centreline. This anomaly could just be a random event, and it is not possible to tell from the limited samples taken at the time.

Trenching – Part 2

A second round of trenching was carried out on 27 November 2003 in order to confirm the original test results obtained, and to also check the change in the base properties with depth. Testing was carried out only on failed sections, with samples being taken at 2.2m and 3.8m from the centreline, over a number of depths. A summary of the results is contained in Appendix E.

Moisture Content Test Results

The moisture content test results from the further materials sampling carried out are summarised in the table below.

As can be seen from the test results, the moisture content in the samples at 2.2m from the centreline was pretty much constant with depth, indicating that moisture was probably not entering the pavement at these locations.

In the samples taken from 3.8m from the centreline, it can be seen that the moisture content was greatest in the top 50mm, indicating that moisture was getting into this top layer, possibly through the surfacing.

Table 4.9 – Moisture content test results from second round of trenching

Location	Distance from centreline (m)	Depth Range (mm)	Moisture Content (%)
Ch 75553	2.2	0 – 50	4.1
		100 – 150	3.9
		250 – 300	4.4
	3.8	0 – 50	5.5
		100 – 150	4.7
		250 - 300	4.9
Ch 75551	2.2	0 – 50	3.8
		100 – 150	4.1
		200 - 250	3
		250 – 300	4
	3.8	0 – 50	5.3
		100 – 150	5
		200 - 250	3.5
		250 - 300	3.7

Particle Size Distribution (Grading)

This was tested for the top 50mm of the base material. The results were compared to the required specifications and the most recent original quarry stockpile test results.

As can be seen, the results of the grading testing indicate that the grading was changing in the top 50 mm with more fines being present. Interestingly, in contrast to the previous grading results, more fines were generally present at 2.2 m from the centreline, not at 3.8m where the failure is occurring.

Figure 4.5 – Trenching Part 2:

a) A view of the trenches excavated in the second round of testing

b) A close view of one of the holes, showing the moistness in the base material

Table 4.10 – Grading test results from second round of trenching, compared to quarry results

		11/11/02	27/11/03: Results for top 50mm	
AS Sieve Size (mm)	**Specified C Grading**	**Quarry**	**2.2m from CL**	**3.8m from CL**
37.5	100	100	100	100
19	80 – 100	99	98.5	99
9.5	55 – 90	64	68.5	66
4.75	40 – 70	47	52.5	46.5
2.36	30 – 55	31	36.5	30.5
0.425	12 – 30	14	18.5	17.5
0.075	5 – 20	8.1	11.5	13

These test results seemed to confirm that some breakdown or segregation of the gravel in the base was occurring, but the material still met the specifications. With no testing having been done to assess whether the material was prone to breakdown, it was difficult to assess whether this was a contributing cause of the failure.

Other Test Results

These tests were done for the top 50 mm of the base material, and for the depth 200-250mm. The results were compared to the required specifications and the most recent original quarry stockpile test results.

From the results, it can be seen that all of the properties had increased in both samples, and that the greatest increase in the properties had occurred at 3.8m from the centreline, where the failure was occurring. This result was expected.

Also, it is seen that the magnitude of the properties is greater for the top 50mm, than for the depth range 200-250mm, indicating that the problems with the pavement are occurring close to the surface.

Table 4.11 – Other test results from second round of trenching, compared to quarry results

	Specifications	Quarry	27/11/03: Average Values for top 50 mm (200-250 mm)	
Test	**MRS11.05 – Type 3.1**	**11/11/02**	**2.2m from CL**	**3.8m from CL**
Liquid Limit (%)	25 max	21.4	23.9 (22.4)	27.3 (24)
Plasticity Index (%)	6 max (6-8.5)	5.6	8.1	12.7
Linear Shrinkage (%)	3.5 max (3-5)	4	4.7 (4.2)	7.5 (4.6)
Plasticity Index x % < 0.425mm	150 max	81	151	226
Linear Shrinkage x % < 0.425mm	85 max	58	87.5	134
Ratio 0.075/0.425	0.35 – 0.55	0.56	0.62	0.73

The liquid limit values had increased, but almost met the specifications. All of the other values had increased markedly and fail to meet the specifications, especially at 3.8m from the centreline.

The above results seemed to confirm that the problem with the pavement was occurring in the top 50 mm of the pavement, especially when considering the previous testing carried out over a 150 mm depth, where the properties had increased, but only a fraction as much.

Summary of Test Results

Moisture

Moisture contents were higher in the failed sections than the non-failed sections. Between the wheelpaths, moisture content was generally constant throughout the depth of the base. Near the outer wheelpaths where the failure was occurring, moisture content was greatest just below the surfacing, and decreased with depth.

Particle Size Distribution (Grading)

Grading changed in the failed pavement sections, with increased fines, compared to the original grading. The greatest change occurred in the top 50mm, where there was a significant increase in the amount of fines. Grading still met the specifications, despite the changes.

Other

Liquid limits, plasticity index, linear shrinkage and other associated values had increased in the failed pavement sections. The change was greatest in the top 50 mm of the base, where the values had increased markedly.

Between the wheelpaths, there was been a moderate increase, leading to most of the specification values being exceeded or close to it. Near the outer wheelpaths, where the failures are occurring, the increase was very great, with all specification values being exceeded.

4.1.6 Determine Probable Cause(s) of Failure

The above section of the report details all information regarding the project, the failures occurring and the investigation and testing carried out.

Next all possible failure causes were considered, along with any information that either supports or refutes each hypotheses. Since the failure occurred first as bleeding and flushing and rutting is starting to occur, the possible cause(s) of each of these failure types will be examined.

Table 4.12 – Possible failure causes for bleeding and flushing, with associated information

Possible Failure Cause	Information
Too much binder sprayed	Heavy spray rate in seal was placed over bleeding already occurring in primer seal
Insufficient surface aggregate applied	Not observed during visual investigation
Non-uniformity/patching of original surfacing, leading to rise of binder	Unlikely, since surfacing was reconstructed before sealing took place
Embedment of surface aggregate, due to weakness of base layer below	Grader made tears in surface, and these were not fixed up before sealing. These were some of areas that bled up. Also grading and other testing indicates that some breakdown of top 50 mm of base material may be occurring, giving adverse plasticity and shrinkage.
Lack of proper rolling during placement	No evidence
Failure to protect newly constructed surface from traffic for long enough	No evidence, although lack of trafficking prior to sealing may have influenced extent of failure, with more occurring in westbound lanes
Loss of surface aggregate due to stripping or ravelling	Not observed during visual investigation
Breakdown of surface aggregate	No evidence, although breakdown of base material may be occurring
Poor spreading of aggregate	Not observed during visual investigation
Over-filled voids in asphalt	Not applicable
Lack of size of aggregate (due to grading), leading to being covered by binder	No evidence, but may be partly true, although problem is with excess binder being sprayed, not with incorrectly sized aggregate being used.

Table 4.13 – Possible failure causes for rutting, with associated information

Possible Failure Cause	Information
Inadequate pavement thickness	No evidence, since pavement was not failing previously when a thinner depth
Weak subgrade (adjacent bulging of pavement or shoulder surface)	No evidence, since subgrade is known to be weak and this was taken into account in design
Weak base	Grading and other tests indicate that plasticity and shrinkage has increased in top 50mm, possibly also leading to weakness or low CBR
Surfacing lack of strength / stability	No evidence, and surfacing is fairly thin anyway, and is not designed to fulfil a structural function
Inadequate Compaction	No evidence, since compaction test results are okay
Poor Material Quality	Original quarry tests are okay, but materials may have deteriorated in service, probably due to moisture entry
Excessive Moisture	Testing indicates moisture is high, especially near failures and in top 50 mm of base
Very high traffic loading (> 10^7 ESAs)	Design traffic is 3.8 x 10^5, and not excessive
Inappropriate mix design (asphalt)	Not applicable
Interaction between layers (e.g. excess cutter moving into next layer)?	Interaction between primer seal and seal, where excess binder in primer seal flushed up through seal

Probable Cause(s) of Failure

From the above tables, it can be seen that several possible broad failure cause(s) have been identified. These probable failure causes and contributing factors will be discussed below.

Too much binder was sprayed in the final seal, not taking into account the existing texture of the primer seal at various locations, especially the outer wheelpath area where the failures were occurring. The excess binder in the primer seal subsequently migrated up through the seal to the surface.

Embedment of surface aggregate, due to weakness of base layer below, was probably caused by a lack of time for densification and trafficking prior to sealing, and tears made by a grader in the surface not being repaired.

There was a weak base, with questionable material quality. While quarry results were satisfactory, in-situ values indicated possible breakdown or segregation of the material with an increase in fines, leading to high plasticity and shrinkage, especially when the moisture content and degree of saturation increased. This was mostly true in the top 50mm. The rest of the pavement structure appeared to be okay.

There was excessive moisture is in the pavement, which may be the possible cause of the poor material quality and degradation. Since the top 50 mm of the pavement is experiencing the highest moisture values, it was suspected that the moisture was being forced through the seal by heavy vehicles.

Since in this case the failure causes have been identified fairly easily (but not conclusively) due to the extensive investigation and testing carried out, the matrix method for identifying the probable failure causes was not required to be used.

Learning from the Failure

From the above, it can be seen that an improved construction process may have reduced the severity and magnitude of the failures. The following has been observed during the investigation:

The process of laying the base material has met the specification requirements, but seems to have contributed to the early embedment of the surface aggregate. The base could have benefited from a longer finishing process involving more chance for densification and 'baking out' to form a hard surface.

The unbound pavement, although meeting specifications, is loose and deforms under traffic once moisture contents increase. The integrity of the seal is important for the long-term performance of the pavement, especially having good surface texture.

The primer seal did not appear to have bound the pavement surface together, or provided impermeability in the pavement surface.

The seal appears to become permeable, and allows water to be forced through, once the binder level rises to near the top of the surface aggregate. This problem was increased by the choice of stone size used for the primer seal and seal.

The 10 mm primer seal aggregate had good texture, except where flushed up. The quantity of binder required to hold the 14 mm seal aggregate on a well-textured 10 mm surface was high, and this rate became much too great when sprayed on top of a flushed primer seal surface with little surface texture.

The degree of saturation is related to both the moisture content and density. Initially, the degree of saturation was satisfactory. However, as moisture entered the pavement and it densified under traffic loading, the degree of saturation could be expected to rise dramatically, contributing to the failure.

The increase in fines and degradation in properties is mostly confined to the top 50 mm of the base and is probably due to a combination of moisture entering through the surface and loading from heavy traffic.

The original testing, done to a depth of 150mm, failed to completely highlight the problem. Testing over 50 mm thicknesses helped reveal the problem.

Suggestions for how the construction process could be improved to help prevent this type of failure in the future include the following:

Construction of unbound granular pavements under this type and volume of traffic should be avoided. Sidetracks may be required to allow sufficient surface densification using rollers and watering techniques. Ball penetration tests should be used more as a guide to surface hardness. If a sidetrack is used, a prime could be used, instead of a primer seal, otherwise a tougher surfacing such as asphalt may be required.

A compaction standard of greater than 100% e.g. 102%, would be useful to help prevent further densification under traffic occurring. This applies to National and State Highways with high traffic volumes.

Stone sizes for the sealing treatments should be selected in accordance with local guidelines, or other guidelines as appropriate.

The time between the initial surface treatment and the seal must be long enough to allow trafficking to reveal any weakness in the pavement, and allow orientation of the aggregate in the initial treatment.

Edgelines may be required on the initial surface treatment to help ensure that traffic travels in the correct location, and surface texture issues are not as complicated. Variable spray rates may also be required across the pavement profile.

A supplementary specification for surface finishing would help to provide quality in this area, and an item should be specified in the schedules, being paid per square metre.

4.1.7 Determine Best Rehabilitation Treatment

Possible Rehabilitation Treatments

A study of possible rehabilitation options was conducted. These options should have met the earlier recommendation that work under traffic be avoided, unless a robust sealing alternative such as asphalt was selected.

Table 4.14 - Possible rehabilitation options for 67/18B/302.1 – Bowenville-Dalby

Treatment Option	Project Cost Estimate ($)	Construction Risk	Long Term Benefit
Full width asphalt overlay	531,000	Low	High
Full width stabilisation	280,000	Medium	Medium
Full width granular overlay	335,000	High	High
Stabilise outer wheelpaths	210,000	Medium	Medium
Asphalt in outer wheelpaths	310,000	Low	High
Granular material in outer wheelpaths	260,000	Medium	High

The length of work required was almost the whole 1.5 km length of the job, with failures being concentrated in the outer wheelpaths. However, it was possible that failure may also occur in the inner wheelpaths in the future.

Full width asphalt overlay

A 80 mm thick structural layer on top of existing pavement would be probably be satisfactory. This treatment option acknowledges that the gravel has a structural deficiency. Due to the deformation occurring in the existing pavement, about $60,000 of prior works such as surface correction and pavement repairs would probably be needed.

The risks associated with this option are very low. There should be no traffic management problems, due to the relative speed of the asphalt placement, and minimal weather risks. The asphalt may be prone to fatigue, and so some sort of polymer-modified binder may need to be used to address this. This option may not be possible due to grade line restrictions and hydraulic effects such as afflux.

This option should provide a very good pavement life, and would provide a consistent pavement over the full road width. An open graded asphalt surfacing e.g. OG10, may be used to prevent water entering the pavement under pressure. Minimal future maintenance would be required, with no need for the scheduled second seal

Table 4.15 - Cost estimate: full width asphalt overlay

Item	Amount	Details
Unit Costs		
DG Asphalt ($/$m^3$)	336	Density = 2.4 ton/m^3 Cost = $140/ton
Quantities (m)		
Length	1500	Whole length of job
Width	10	Excludes outermost shoulder area
Depth	0.08	80 mm thick overlay
Costs ($)		
DG Asphalt	405,000	Volume x Unit Cost
Prior works	60,000	Surface correction and pavement repairs
Contract cost	465,000	DG Asphalt + Prior works
Project cost	531,000	Allows 14% for supervision and overhead costs

Full width stabilisation

Stabilisation to a depth of about 200 mm should 'improve' the gravel and help address the higher linear shrinkage and plasticity found during testing. The type of agent to be used for the stabilisation would be confirmed after testing, but would probably consist of a lime and fly ash mix.

The risks associated with this option are low to medium. There should be minimal traffic management problems, as the agent will allow the work to be done under traffic, and risks due to weather should be minor. There is a chance of cracking occurring, due to the increased rigidity and brittleness caused by the stabilisation. Ride quality may not be as good as desired due to restricted work time, and rutting will occur if adequate strength is not achieved.

This option should produce a good pavement life, with a consistent pavement over the full road width. A fabric seal placed on top after completion would reduce the chance of water entering the pavement under pressure, and cracking. Therefore, the scheduled second seal would not be needed.

Table 4.16 - Cost estimate: full width stabilisation

Item	Amount	Details
Unit Costs		
Area Cost ($/m^2)	18	Covers stabilisation, primer seal and fabric seal
Quantities (m)		
Length	1500	Whole length of job
Width	9	Excludes outermost shoulder area
Costs ($)		
Contract cost	243,000	Unit Cost x Area
Project cost	280,000	Allows 14% for supervision, overhead costs

Full width granular overlay

To meet with the recommendations learned from the failure, either a sidetrack or an asphalt surfacing would be required, to prevent occurrence of a similar failure. No prior works would be required, since the 150 mm overlay would improve the structural capacity of the failed areas.

The risks associated with this option are medium to high. Construction would take long time (compared to asphalt overlay or in-situ stabilisation) and wet weather could cause problems.

Table 4.17 - Cost estimate: full width granular overlay

Item	Amount	Details
Unit Costs		
Granular material ($\$/m^3$)	70	
OG Asphalt surfacing ($\$/m^3$)	322	Density = 2.3 ton/m^3 Cost = $140/ton
Quantities (m)		
Length	1500	Whole length of job
Width: granular overlay	11	Whole road width
Depth: granular overlay	0.15	150 mm thick granular overlay
Width: open graded asphalt	8.2	Not including most of shoulders
Depth: open graded asphalt	0.03	30 mm open graded surfacing
Costs ($)		
Granular overlay	173,250	Volume x Unit Cost
OG Asphalt	119,000	Volume x Unit Cost
Contract cost	292,250	Granular Overlay + OG Asphalt
Project cost	335,000	Allows 14% for supervision, overhead costs

This option may not be possible due to grade line restrictions and hydraulic effects such as afflux. Overnight traffic may cause damage to the unsealed pavement, and while not a long term problem, it would slow down the work.

This option should give a good pavement life using a granular overlay as originally designed. An open graded asphalt surfacing, placed on top of the seal, would help to prevent water entering the pavement under pressure. The scheduled second seal would not be needed.

Stabilise outer wheelpaths

The in-situ stabilisation (to a depth of about 200mm) of the material in the outer wheelpaths may prevent the high linear shrinkage and plasticity values occurring in the gravel here. The type of agent would be confirmed after testing, but would probably consist of a lime and fly ash mix. The scheduled second seal would still be required after this work was completed.

Table 4.18 - Cost estimate: stabilise outer wheelpaths

Item	Amount	Details
Unit Costs		
Area Cost ($\$/m^2$)	18	Covers stabilisation, primer seal and fabric seal
Quantities (m)		
Length	1500	Whole length of job
Width	2.4	Outer wheelpath, one way
Costs ($)		
Contract cost	129,600	Unit Cost x Area (x 2: both ways)
Cost + Overheads	150,000	Allows 14% for supervision, overhead costs
Cost of second seal	60,000	
Total project cost	210,000	Cost + Overheads + Cost of second seal

The risks associated with this option are low to medium. There should be minimal traffic management problems, as the agent will allow the work to be done under traffic, and risks due to weather should be minor. There is a chance of cracking occurring, and so a fabric seal may be required.

There is a possibility that the inner wheelpaths may fail some time in the future. Rutting will occur in the outer wheelpaths if adequate strength is not achieved. This option will result in an inconsistent pavement across the road width.

Providing adequate strength is achieved and the inner wheelpaths do not fail, this option should provide a good pavement life, while being cost effective.

Asphalt in outer wheelpaths

This option involves milling out the outer wheelpaths and replacing the material with 80 mm of dense graded asphalt e.g. DG20. This option addresses the failed wheelpaths, and construction should be possible under traffic. The scheduled second seal would still be required after this work was completed.

The risks associated with this option are very low. There is no risk of cracking, and construction should be quick to complete. The work will result in an inconsistent pavement across the road width, and there is still a chance that the inner wheelpaths may fail in a similar manner to the outer wheelpaths.

This option would give high strength outer wheelpaths. It is a cost-effective solution that targets the failed areas.

Table 4.19 - Cost estimate: Asphalt in outer wheelpaths

Item	Amount	Details
Unit Costs		
DG Asphalt ($/m^3)	350	Density = 2.5 ton/m^3 Cost = $140/ton
Milling ($/m^2)	5	
Placement ($/m^2)	3.5	
Quantities (m)		
Length	1500	Whole length of job
Width	2	Outer wheelpath, one way
Depth	0.08	80 mm thick milling and asphalt
Costs ($)		
DG Asphalt	168,000	Volume x Unit Cost (x2: both ways)
Milling	30,000	Area x Unit Cost (x2: both ways)
Placement	21,000	Area x Unit Cost (x2: both ways)
Contract cost	219,000	DG Asphalt + Milling + Placement
Cost + Overheads	250,000	Allows 14% for supervision, overhead costs
Cost of second seal	60,000	
Total project cost	310,000	Cost + Overheads + Cost of second seal

Granular material in outer wheelpaths

This option involves tyning up the material in the outer wheelpaths with a slight addition of a stabilising agent, respreading and recompacting, then sealing and resurfacing with open graded asphalt. This should provide a robust surfacing and the pavement gravel should meet the specifications, due to the addition of the additive to allow for possible construction deficiencies and the shrinkage problems revealed during testing.

The risks associated with this option are low to medium, due to the minor pavement modification. Construction under traffic should not be a problem with the asphalt surfacing.

Table 4.20 - Cost estimate: Granular material in outer wheelpaths

Item	Amount	Details
Unit Costs		
OG Asphalt surfacing ($/m^3)	322	Density = 2.3 ton/m^3 Cost = $140/ton
Tyning and Sealing ($/m^2)	15	
Quantities (m)		
Length	1500	Whole length of job
Width	2.4	Outer wheelpath, one way
Depth	0.03	30 mm thick asphalt surfacing
Costs ($)		
OG Asphalt	119,000	Volume x Unit Cost (x2: both ways)
Tyning and sealing	108,000	Area x Unit Cost (x2: both ways)
Contract cost	227,000	OG Asphalt + Tyning and Sealing
Total project cost	260,000	Allows 14% for supervision, overhead costs

This cost effective option should provide good outer wheelpath protection, with the open graded surfacing helping to prevent water entering the pavement under pressure. The pavement would be reasonably consistent across the road width, and the scheduled second seal would not be required afterwards.

Follow up after rehabilitation

While the investigation and testing discussed above were being carried out, the road section continued to deteriorate, until in about November 2003 the failures were occurring along the whole length of the project in both directions, still only in the outer wheelpaths. The magnitude of the pavement deformation in these affected areas also continued to increase.

In June 2004, near the end of the financial year, funds became available, and so rehabilitation work was carried out. The rehabilitation treatment consisted of a treatment similar to option 6 above, where the outer wheelpaths were tyned, a stabilisation agent added, the material was respread and compacted, with a surfacing being placed on top.

In addition, a second seal will probably be placed in the future. This will probably be a fabric seal, providing an impermeable membrane to prevent moisture entry through the surface in the future.

The road section was measured for roughness and rutting as part of the annual network level survey, soon after the rehabilitation work was completed in June 2004. This enables the effectiveness of the treatment, at least in the short-term, to be evaluated.

Table 4.21 – Roughness and average rutting over the length of the job in June 2004, compared to the average for the road section

Condition Measure	Roughness (counts)	Average Rutting (mm)
Average: Ch. 42 - 84	85	5.2
Ch. 75 – 76	47	3.3
% of Average	55	63

The average measurements over the range 75 – 76 km cover 1 km of the job length and these values can be compared to the average values over the second half of the Warrego Highway, between Toowoomba ands Dalby, from Chainage 42 – 84 km. These comparisons are shown in the table.

It can be seen that the rehabilitation work has been successful in reducing the roughness and rutting, at least over the short-term. The road section is currently one of the least rough and rutting sections along this 42 km part of the Warrego Highway between Toowoomba and Dalby.

Whether this will continue to be the case depends very much on how successful the selected rehabilitation treatment was in treating the cause of the initial pavement failure.

4.2 Brookstead-Pampas

4.2.1 Plan the Investigation

General Review of the Problem

The project 81/28A/301 was constructed on the Gore Highway (28A) in the region near Brookstead and Pampas, and was completed in January 2001. It involved the reconstruction of the sections of the pavement between Chainages 57.89 and 62.62 km, a nominal length of 4.73 km.

The work involved the replacement of the existing pavement and resurfacing, with the work varying along the length of the project, depending on the existing pavement condition. The details about this will be discussed in a later section.

Three and a half years after completion, there is considerable bleeding in the wheelpaths along the length of the job, and excessive rutting and roughness may also be occurring. This report will investigate the extent of the failures, possible failure causes, and the need for rehabilitation work.

Scope of Work

Importance of the road

The Gore Highway between Toowoomba and Millmerran (28A) is part of the National Highway system and an important road link for the Darling Downs. As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

The traffic volumes on this road are high for a rural road. In 2003, the AADT (Average Annual Daily Traffic) in the area of the job was about 2210 vehicles/day, including about 645 heavy vehicles, composing about 29% of the total traffic volumes. These heavy vehicles include a significant number of Type 1 Road Trains and B-Doubles, in addition to Semi-Trailers. Hence, the use of the road for the transport of freight is very important.

Figure 4.6 – Brookstead – Pampas:

a) A view of the considerable bleeding in the wheelpaths

b) A closer view of some bleeding in the outer wheelpath. Rutting is also starting to occur.

Magnitude of the failure

The current extent of the failure (if any) occurring over the length of this project can be assessed by examining asset management data, available from the ARMIS system.

For simplicity, only roughness and average rutting data were examined, with the data being averaged over 1 km intervals. Surface texture and skid resistance is unlikely to be important in this rural area with few intersections; however the extent may influence the speed of other failures, such as rutting. The results of the statistical analysis of these data points is summarised in the following table:

Table 4.22 – Roughness and average rutting over the Brookstead-Pampas section, compared to the average for the road section (40-79 km)

Chainage (km)	Average Roughness (count)	% of 40-79	Average Rutting (mm)	% of 40-79
40 - 79	**74**	**100**	**4.4**	**100**
58 - 59	65	88	6.4	145
59 – 60	94	127	4.3	98
60 – 61	54	73	3.2	73
61 – 62	52	70	2.7	61
62 – 63	56	76	3.1	70

The following can be seen from data in the above table:

- 58 – 59 has roughness below average, but very high rutting
- 59 – 60 has high roughness, and rutting close to average
- 60 – 63 (three sections) all have roughness and rutting below the average

So, it can be concluded that the worst failure is concentrated in the early part of the job, between chainages 57.9 – 60 km. The later part of the job, from 60 – 62.62 has much better performance.

The above analysis, using values averaged over 1 km intervals, obviously cannot take into account failures occurring over a much smaller length, but provides a good overview of the overall performance of the road section.

Current risk to road users and risk of deterioration in the future

The current risk to road users is minimal. This failure is more one that is failing to meet long-term performance expectancies, rather than one that is failing in a sudden and dangerous manner. The pavement may be expected to deteriorate further in the future unless a rehabilitation treatment is applied. However, the rate of deterioration would be expected to be minimal, and able to be catered for by the normal road maintenance process.

Investigation Plan

The goal of the investigation was to assess the failure magnitude, determine possible causes of the failure and the need for rehabilitation work. The investigation consisted of a review of asset management data, documents and literature, and a brief visual investigation.

4.2.2 Review Documents and Literature

Plans and Pavement History

The general work and pavement history carried out in this project is summarised as follows, and was mainly found from the plans and the ARMIS system. All work is listed from the bottom of the pavement up. Sections shaded in grey are those where major pavement reconstruction was undertaken.

As can be seen, the work varied considerably along the length of the job, depending on the existing pavement condition. In some sections, the whole pavement structure was replaced, while in other places it was left in place and only the sealing treatment was applied.

Over all of the length a 16 mm spray seal was applied as the final treatment. A 10 mm primer seal was applied before the spray seal along most of the job.

Table 4.23 – Summary of work carried out on Brookstead-Pampas: 81/28A/301

Chainage Range (km)	Length (km)	Work carried out	Existing material left in place
57.89–58.1	0.21	16 mm seal	125, 2 x 150 mm granular, 16 mm seal (1993)
58.1–58.51	0.41	175, 2 x 150 mm granular 10 mm primer seal 16 mm seal	Nil
58.51–59.33	0.82	250, 2 x 150 mm granular 10 mm primer seal 16 mm seal	Nil
59.33–59.93	0.60	300, 2 x 150 mm granular 10 mm primer seal 16 mm seal	Nil
59.93–59.96	0.03	16 mm seal	2 x 100 mm granular, 20 mm seal (1979) 20 mm seal (1988)
59.96–61.38	1.42	225, 2 x 150 mm granular 10 mm primer seal 16 mm seal	Nil
61.38–62.45	1.07	10 mm primer seal 16 mm seal	2 x 100 mm granular, 20 mm seal (1979)
62.45–62.55	0.1	225, 2 x 150 mm granular 10 mm primer seal 16 mm seal	Nil
62.55–62.6	0.05	16 mm seal	2 x 100 mm granular, 20 mm seal (1979)

A summary of the extent of the reconstruction work for 1 km sections is shown below. New pavement refers to that placed in during the job, while old pavement refers to that placed prior to 2001, and not removed.

Table 4.24 – Extent of reconstruction work: 1 km sections – Brookstead-Pampas

	New Pavement		**Old Pavement**		
Ch. (km)	**Length (m)**	**Depth (mm)**	**Length (m)**	**Depth (mm)**	**Year**
58 - 59	410 490	475 550	100	425	1993
59 - 60	330 600 40	550 600 525	30	200	1979
60 - 61	1000	525	-	-	-
61 - 62	380	525	620	200	1979
62 – 62.6	100	525	500	200	1979

It is useful to compare the roughness and rutting observed over each section with the average seal and pavement age. All of this information, available from the ARMIS system, is summarised below. The final row includes the range 62 – 63 and so 400m outside the job range, the effect of this would be minimal.

As can be seen, the first three 1 km sections of the job consist of fairly new pavement, while the last 1.6 km has a much older average pavement age. Despite their low pavement age, the first two sections have very high roughness or rutting, while the older sections (61 – 62.6) have much lower roughness or rutting.

Table 4.25 – Comparison of roughness and rutting with average seal and pavement ages: Brookstead-Pampas

	% of Average: 40-79		Age (years)	
Chainage (km)	**Roughness**	**Rutting**	**Seal**	**Pavement**
58 – 59	88	145	3.5	4.3
59 – 60	127	98	3.5	4
60 – 61	73	73	3.5	3.5
61 – 62	70	61	3.5	17.1
62 – 63	76	70	7.4	18.4

From this, it could probably be concluded that the failure is not due to an aged pavement structure. Instead the failure could be due to specific environmental conditions that have a greater effect towards the start of the job, or some problem with the newly reconstructed pavement sections.

4.2.3 Non-destructive Condition Survey

Visual Examination

A visual investigation of the pavement section was conducted in July 2004. While only of a superficial nature, it did provide some useful information.

The failure consisted of flushing/bleeding of the pavement, mostly within the wheelpaths, with rutting also beginning to increase in some places. In addition, some roughness was noted to be due to isolated failures, such as depressions. The following information regarding the flushing/bleeding was noted.

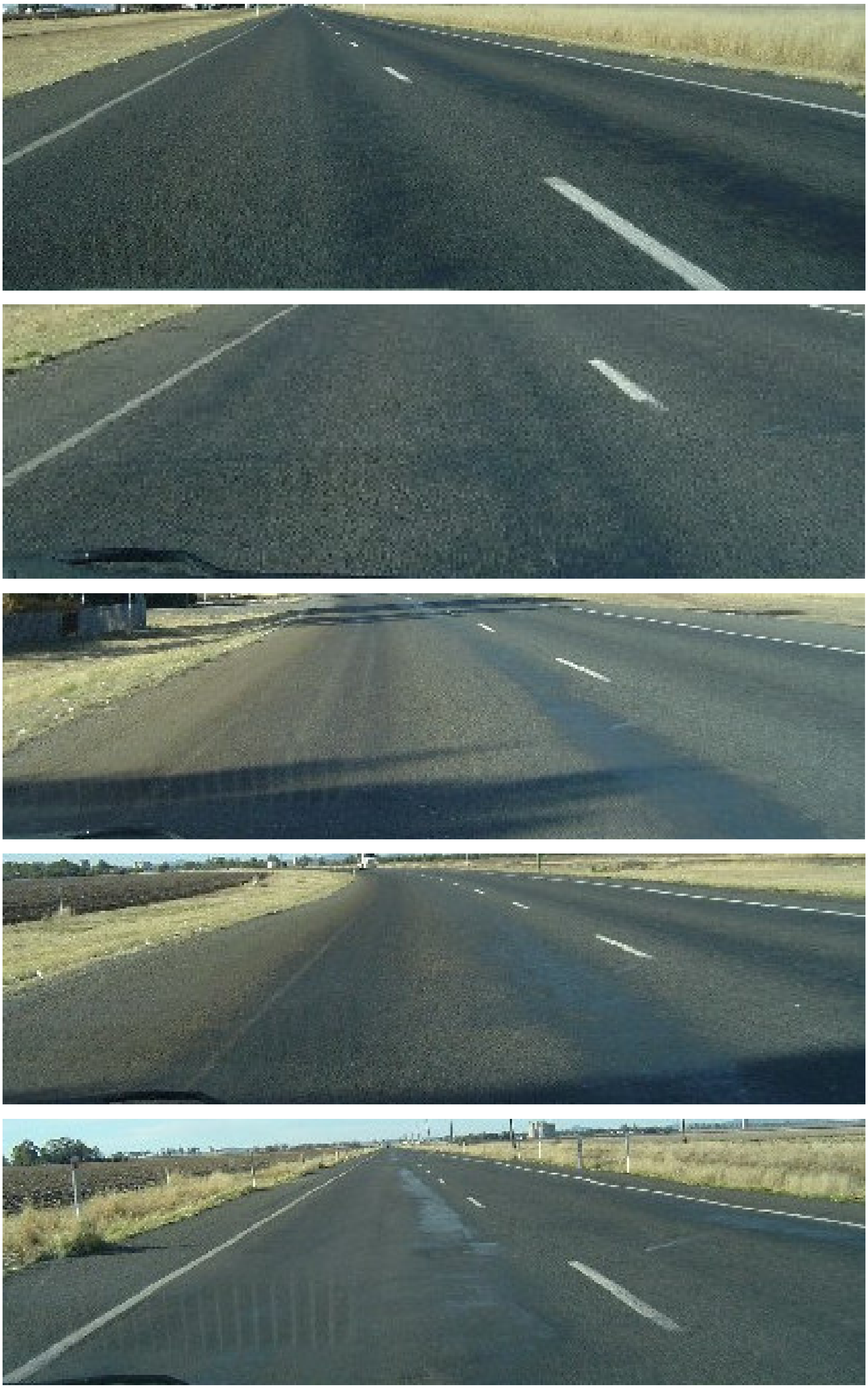


Figure 4.7 – A series of pictures illustrating the variability of bleeding along the section

The bleeding was occurring randomly throughout the length of the job, with length of affected sections varying from a few metres up to a hundred metres or more. It was occurring on both sides of the road, and was distributed within the wheelpaths, and contained within the edgelines.

In some sections, only the outer wheelpath was affected, in others only the inner wheelpath, while in other sections both wheelpaths were affected.

The road goes through a flat floodway area, with agricultural land either side of the road. Therefore, it is possible that during heavy runoff, moisture could have moved into the pavement, contributing to the failures that are occurring. However, no specific failures of the drainage system were observed.

4.2.4 Determine Probable Cause(s) of Failure

The above section of the report details all information regarding the project, the failures occurring and the investigation and testing carried out. Next all possible failure causes were considered, along with any information that either supports or refutes each hypotheses.

Since the failure is mainly occurring as bleeding and flushing, the possible cause(s) of this failure type will be examined.

Table 4.26 – Possible failure causes of bleeding and flushing, along with associated information

Possible Failure Cause	**Information**
Too much binder sprayed	No evidence
Insufficient surface aggregate applied	Not observed during visual investigation
Non-uniformity/patching of original surfacing, leading to rise of binder	Unlikely, since pavement was reconstructed in most places, before sealing took place.
Embedment of surface aggregate, due to weakness of base layer below	May be the most likely cause of the bleeding. Could be due to weakness of base layer due to moisture entry or other factors
Lack of proper rolling during placement	No evidence
Failure to protect newly constructed surface from traffic for long enough	No evidence
Loss of surface aggregate due to stripping or ravelling	Not observed during visual investigation
Breakdown of surface aggregate	Not observed during visual investigation
Poor spreading of aggregate	Not observed during visual investigation
Over-filled voids in asphalt	Not applicable
Lack of size of aggregate (due to grading), leading to being covered by binder	No evidence

Probable Cause(s) of Failure

From the above, it can be seen that based on the rather limited information available, the most likely cause of the bleeding/flushing is embedment of surface aggregate, due to a weakness of base due to moisture entry or a lack of surface finishing.

The location of the bleeding/flushing in the wheelpaths indicates that it is not probably due to accidentally spraying excess binder. It does support the theory that it is related to aggregate embedment under traffic loading.

The worst roughness and rutting is occurring in the newly reconstructed sections, and this possibly indicates that possible causes are conditions specific to the reconstruction, such as a weak base causing surface aggregate embedment.

Conclusion

Based on the rather limited information gathered for the investigation, it is likely that the bleeding is due to aggregate embedment. This could have been caused by moisture entry into the pavement.

However, the worst roughness and rutting is occurring in sections with new pavement, indicating that something else contributed to the deterioration of the pavement that caused this.

It is possible that when the sections were reconstructed, there was a lack of surface finishing applied, resulting in a surface that would allow easy aggregate embedment under the traffic volumes occurring on the road. In addition, there may have been a lack of time for the pavement to densify under traffic before the final seal was applied.

4.2.5 Determine Best Rehabilitation Treatment

Possible Rehabilitation Options

Since the failure is still fairly minor, the need for rehabilitation work at present is not urgent. However, rehabilitation may be required in the future, if the section continues to deteriorate. A list of possible rehabilitation options to treat the bleeding/flushing is as shown below. Since the rutting is currently not at an excessive level, there is not a great need to correct it at present.

The choice of which rehabilitation treatment would be best to use could be determined by further investigation in the future, when the pavement has deteriorated to a level that justifies retreatment.

Overlay with open graded asphalt

This option could be used to absorb the excess binder, unless hardening has occurred. In the flat country where the failure is occurring, this option could be limited by afflux and hydraulic considerations.

Slurry seal

This treatment could be effective at present, but if further deterioration occurs and additional structural capacity is required, it would be of no use.

Bitumen reseal

This treatment could be successful if the binder rate was adjusted down, to allow excess binder to be absorbed. Control of traffic in early stages would be important to prevent stripping.

In-situ stabilisation

This may give additional strength to the base, reducing the chance of failure due to surface aggregate embedment in the future. However, since pavement is still structurally sound, it is probably an excessive treatment, unless further deterioration of the pavement structure occurs.

High pressure water retexturing

This option is roughly double the cost of a reseal. However, it is likely to be more effective in a long-term reduction of the bleeding and flushing that is occurring.

4.3 Drayton Connection Road

4.3.1 Plan the Investigation

General Review of the Problem

The Drayton Connection Road (321) connects the Gore Highway (28A) and the New England Highway (22B) and is a total length of 11.2 km. Sections of the road are undergoing deterioration causing excessive roughness.

Some sections have been rehabilitated in the past using in-situ stabilisation patching of the pavement. However, these sections have continued to deteriorate, and more stabilisation will probably be carried out in the future. This report will investigate the extent of the failures, possible failure causes, and the need for rehabilitation work.

Scope of Work

Importance of the road

The Drayton Connection Road is designated as a regional state-controlled road, with funding provided by the State Government, and all work being administered by the Queensland Department of Main Roads.

The traffic volumes on this road are fairly high. In 2003, the AADT (Average Annual Daily Traffic) on the road was about 3350 vehicles/day including 410 heavy vehicles comprising about 12% of the total traffic volumes. These heavy vehicles are mostly trucks/buses, semi-trailers and articulated vehicles.

Magnitude of the failure

The current extent of the failure (if any) occurring over the length of this road can be assessed by examining asset management data, available from the ARMIS system.

Figure 4.8 – Drayton Connection Road:

a) A patch where in-situ stabilisation was carried out. Failure is still occurring.

b) A patch that has failed through potholing, necessitating further patching work, and resulting in a rough surface

For simplicity, only roughness and average rutting data will be examined, with the data being averaged over 1 km intervals. Surface texture and skid resistance is unlikely to be important in this rural area with few intersections; however the extent may influence the speed of other failures, such as rutting.

The results of the statistical analysis of these data points is summarised in the following table, which shows a comparison of the roughness and average rutting for 1 km sections along the road, compared to the average of all values along the road, chainage 0 – 11.2 km. Obviously, the higher the percentage of the average, the worse the road section is, compared to the whole road range.

Table 4.27 – Roughness and rutting data over 1 km intervals, and compared to the overall average: Drayton Connection Road

Chainage (km)	Average Roughness (counts)	% of 0 - 11.2	Average Rutting (mm)	% of 0 - 11.2
0 – 11.2	**66**	**100**	**4**	**100**
0 – 1	52	79	2.8	70
1 – 2	57	86	3.1	78
2 – 3	70	106	4.6	115
3 – 4	83	126	4.9	123
4 – 5	69	105	4.9	123
5 – 6	62	94	3.3	83
6 – 7	55	83	3.5	88
7 – 8	64	97	3.7	93
8 – 9	63	95	3.5	88
9 – 10	65	98	3.2	80
10 – 11	63	95	5.1	128
11 – 11.17	89	135	5.8	145

The following can be seen from the above table:

- 0 – 2 km has roughness and rutting below the average
- 2 – 5 km has roughness and rutting above the average
- 5 – 10 km has roughness and rutting slightly below average
- 10 – 11.17 km has roughness and rutting above the average

The above analysis, using values averaged over 1 km intervals, obviously cannot take into account failures occurring over a much smaller length, but provides a good overview of the overall performance of the road section. It shows that some sections of the road are experiencing considerably greater roughness and rutting than the average, and would probably require rehabilitation work, especially if the routine maintenance required on the sections is high.

Current risk to road users and risk of deterioration in the future

The current risk to road users is minimal. The pavement may be expected to deteriorate further in the future unless a rehabilitation treatment is applied. However, the rate of deterioration would be expected to be minimal, and able to be catered for by the normal road maintenance process.

Planned Future work

Some sections of the Drayton Connection Road will be rehabilitated in the near future using in-situ stabilisation, despite this strategy being less than successful in the past. A lack of testing of material properties may have been responsible for this. It is also possible that this was not the optimal rehabilitation strategy, but instead was selected because the option seemed to be quick and easy, and a quick way to spend excess funding.

Investigation Plan

The goal of the investigation was to assess the failure magnitude, determine possible causes of the failure and the need for rehabilitation work. The investigation consisted of a review of asset management data, documents and literature, and a brief visual investigation.

4.3.2 Review Documents and Literature

Plans and Pavement History

The general work and pavement history along this road is summarised as follows, and was mainly found from the plans and the ARMIS system. The most recently carried out surfacing work is shown in the table below.

Table 4.28 – Most recent surfacing work on the Drayton Connection Road

Chainage (km)	Length (km)	Surfacing Type	Date
0 – 1.2	1.2	Dense Graded Asphalt	Dec 1998
1.2 – 1.54	0.34	Slurry Surfacing	June 1994
1.54 – 2.29	0.75	Dense Graded Asphalt	Dec 1998
2.29 – 3.52	1.23	Spray Seal	May 1997
3.52 – 11.17	7.65	Spray Seal	Feb 1997

General details about the average age of the seal and pavement, and the average pavement depths, are reproduced in the table below.

0 – 2 km (roughness and rutting below the average) has an asphalt surfacing, and comparatively young seal age. 2 – 5 km (roughness and rutting above the average) has a slightly old pavement structure.

10 – 11.17 km (roughness and rutting above the average) has a seal and pavement age similar to the rest of the road. However, the deep pavement depth in this area indicates soil may be comparatively weak, and more prone to the effects of any moisture entry that is occurring.

Table 4.29 – Average age of seal and pavement, and pavement depth along the Drayton Connection Road

Chainage (km)	Seal age (years)	Pavement age (years)	Pavement depth (mm)
0 – 1	5.6	41.4	200
1 – 2	7.1	33.2	284
2 – 3	6.7	31.6	317
3 – 4	7.3	28.4	317
4 – 5	7.4	24.9	300
5 – 6	7.4	24.9	300
6 – 7	7.4	24.9	309
7 – 8	7.4	23.6	390
8 – 9	7.4	23.4	400
9 – 10	7.4	23.4	400
10 – 11	7.4	22.4	434
11 – 11.17	7.4	21.9	450

Published Articles

A document, *Proposed Bulk Maintenance Treatment – Drayton Connection Road* (2003), produced by the Infrastructure Delivery department of Southern District discussed possible bulk maintenance treatments for sections of the Drayton Connection Road, and provided some of the information for this investigation.

The section considered was 4.56 – 5.24 km (a length of 680m), and other possible sections included 3.33 – 3.41 (80m), 9.8 – 10.0 (200m) and 10.27 – 10.48 (210m). It can be seen that these sections are all contained within the regions having high roughness and rutting, as identified earlier from the asset management data.

It was stated that the pavement structure consists of a 100 mm base over a 200 mm laterite subbase, the cost of routine maintenance over the section was roughly 3 times the average for the road, and that regular repairs were required.

The subgrade appeared to be satisfactory, and work that involved digging out the failed pavement (to 300 mm deep) and replacing with stabilised gravel performed well. However, in-situ stabilisation to a depth of 300 mm did not perform well.

It was suspected the failures in in-situ stabilisation patches are probably due to an incompatibility between the laterite subbase and the stabilising blend used.

4.3.3 Interview Personnel

A Materials officer suggested that the choice of in-situ stabilisation as a rehabilitation treatment was mainly due to the perception that it was quick and easy, and a good way to spend some funding. He suggested that if it was to be used as a treatment, much more initial testing should be done, to ensure that the proper blend and type of stabilising agent could be selected.

4.3.4 Non-destructive Condition Survey

Visual Examination

A visual investigation of the pavement section was conducted in July 2004. While only of a superficial nature, it did provide some useful information.

The surfacing of much of the road from chainage 3.3 km onwards was bleeding and flushing within the wheelpaths. In addition, there were locations where the in-situ stabilisation patching work has been carried out.

These patches were observed to be undergoing deterioration such as potholes, rutting and depressions that had been fixed with further maintenance work, indicating that the original work had been unsuccessful in reducing the cost and extent of routine maintenance work required.

Figure 4.9 – Drayton Connection Road:

a) A view of the considerable bleeding that occurs along much of the road

b) Fatigue cracking is starting to occur in this area

4.3.5 Determine Probable Cause(s) of Failure

During the visual investigation, the most significant failures were observed in locations where in-situ stabilisation patching work had been carried out. Without testing having been carried out, it is difficult to determine what the cause of the se failures was.

The suggestion that it was due to an incompatibility between the laterite subbase and the stabilising blend used is very possible. When new stabilised gravel was placed it performed well, whereas in-situ stabilisation of the existing material led to failures, indicating that the problem was due to the existing laterite pavement material. No evidence of poor drainage was found, indicating that this was not the main cause of the problem.

Much of the road is also undergoing bad flushing/bleeding in the wheelpaths. This problem is probably due to surface aggregate embedment into the base, which is of a fairly old age (20-25 years) in most places. While large in extent, the magnitude of this failure is minimal, with the pavement structure appearing to still be sound in the majority of places.

4.3.6 Determine Best Rehabilitation Treatment

The document *Proposed Bulk Maintenance Treatment – Drayton Connection Road* (2003) discussed possible bulk maintenance treatments for sections of the Drayton Connection Road. The discussion was for the section 4.56 – 5.24 km, a total length of 680m. However, the information would also be generally applicable for other sections with failures in the in-situ stabilisation patching.

Possible Rehabilitation Options

The rehabilitation options discussed were as follows:

- Repair of failed sections with maintenance work such as patching and profile correction, followed by a granular overlay.

- Repair of failed sections with maintenance work such as patching and profile correction, followed by an asphalt overlay.

- Minimal repairs followed by in-situ stabilisation of about 100-160 mm of top-up gravel combined with existing base material, followed by a fabric seal. The addition of "new" pavement to combine with the existing would be necessary because of the inconsistent performance of previously in-situ stabilisation of the existing pavement

Selection of Rehabilitation Treatment

The work would probably be carried out under a sole invitee contract, with the local shire, Cambooya, performing the work. Important details considered when determining the best rehabilitation treatment are discussed below, and from considering these, the in-situ stabilisation option would probably be the best rehabilitation treatment.

There was no room for a side-track, and although a granular overlay could possibly be constructed under traffic, it would be risky both from a safety and construction point of view (considering the amount of traffic), as shown by the Bowenville-Dalby granular overlay job, 67/18B/302.1, carried out in this way.

The asphalt overlay option would not be welcomed by Cambooya Shire as it would include a large subcontract and materials components and therefore reduce employment opportunities for the workforce.

The in-situ stabilisation of top-up gravel combined with the base, followed by a fabric seal was considered to be the most suitable treatment. The main issue with this proposed treatment would be to determine a suitable additive for the proposed combination of materials. Considerable up-front testing would have to be carried out to ensure this was achieved.

In-situ Stabilisation Rehabilitation Treatment

Pavement Repairs

It was recommended that the failing pavement be removed and replaced before adding the additional gravel. This was due to the possibility of the failed possibly wet material adversely affecting the long-term performance of the new work.

Shoulder Work

It was recommended that the existing shoulders be replaced with new material to a depth of about 100mm, allowing the full width stabilisation to include the shoulders, reducing the risk of problems in this area.

In-situ Stabilisation

After the additional 160 mm depth gravel had been placed on the surface of the pavement, it was recommended that the in-situ stabilisation work be carried out to a depth of 250mm, including the new gravel and about 90 mm of the existing gravel.

The composition and rate of the additive to be used would have to be determined by further testing, probably unconfined compressive strength tests. This would allow the optimal amount of the best additive to be selected, by finding the combination that gives the highest unconfined compressive strength.

It was recommended that testing be carried out on separate mixtures of new gravel and existing base, new gravel and existing stabilised base, and new gravel. This would help to ensure that an additive and rate suitable for all of the combinations was selected.

Fabric Seal

Within a year of the primer seal being applied, a fabric seal could be sprayed over the full width, to help reduce the risk of cracking of the stabilised material, and reduce the chance of moisture entry through the surface.

4.4 Gatton Bypass

4.4.1 Plan the Investigation

Introduction to the Gatton Bypass

The following sections give some background to the Gatton Bypass and why its construction was necessary. This information was available from the Queensland Department of Main Roads website, http://www.mainroads.qld.gov.au/.

The Queensland Department of Main Roads first recognised the need to construct a bypass of the Warrego Highway (between Ipswich and Toowoomba) around the town of Gatton during the 1960s. The purpose of the bypass was to ease traffic congestion through the town, reduce heavy vehicle movements through residential areas, and provide a quicker travel route for road users.

The Department adopted the current 21 km northern bypass route in 1975 and detailed design started in early 1984. Construction commenced in early 1985 and was completed by November 1989 at a cost of $A23 million.

The project consisted of three major road contracts (including some bridges), two major bridge contracts (Sandy Creek and Lockyer Creek) and one smaller contract for the end connections. The original bypass was designed to accommodate a four-lane upgrade when funding became available. In the meantime however, two overtaking lanes in each direction were constructed in 1993 and 1994.

The existing Gatton Bypass, on the Warrego Highway between Ipswich and Toowoomba, opened to traffic in 1989. The 21 km bypass forms part of the Brisbane-Darwin corridor of the National Highway. Prior to recent duplication work, it was the only section of highway between Brisbane and Toowoomba that did not have four lanes.

The Federal Government announced funding for the duplication of the Gatton Bypass to four lanes in 2001. The Toowoomba office of the Queensland Department of Main Roads managed the duplication project.

The duplication of the Gatton Bypass was necessary due to the high traffic volumes of about 10,000 vehicles/day (including about 2000 heavy vehicles), resulting in a poor accident history.

In the some 15 years that it operated with only two lanes, there were more than 130 reported accidents, with 20 fatalities. The lack of lanes also meant that in the case of a serious accident or maintenance work, traffic had to be detoured through Gatton.

Following from this, the primary objectives for the bypass duplication were to improve safety on the bypass, improve travel conditions and overtaking opportunities, remove the last two-lane section on the Warrego Highway between Brisbane and Toowoomba, and improve intersection safety at the Gatton end.

Duplication of the Bypass

The Gatton Bypass duplication was carried out as three main construction packages each using a slightly different arrangement for design and construction. This enabled construction to start as soon as possible, rather than having to wait for the entire length of the project to be designed.

Package 1, at the eastern end, with a length of 5.1 km, was a design and construct contract, with Bielby Holdings carrying out the work. Package 2, in the centre, with a length of 10.1 km, was designed by RoadTek, and the construction contract was awarded to Stockport. Their financial problems caused some disruptions to the work schedule, and the work had to be completed by RoadTek. Package 3, at the western end, with a length of 5.9 km, was designed by Farr Evrat under contract, with the construction contract being awarded to Bielby Holdings.

The major roadwork involved in the project was the construction of two new lanes on the northern side of the existing road, to become the Brisbane-bound lanes of the bypass. Other work carried out included the modification of the existing lanes from a two-way road to become the Toowoomba-bound lanes of the new bypass (changes to line marking, guardrail installations and signs), duplicating the Helidon interchange at the western end, building a new Gatton-Esk interchange at the eastern end, and other bridge works.

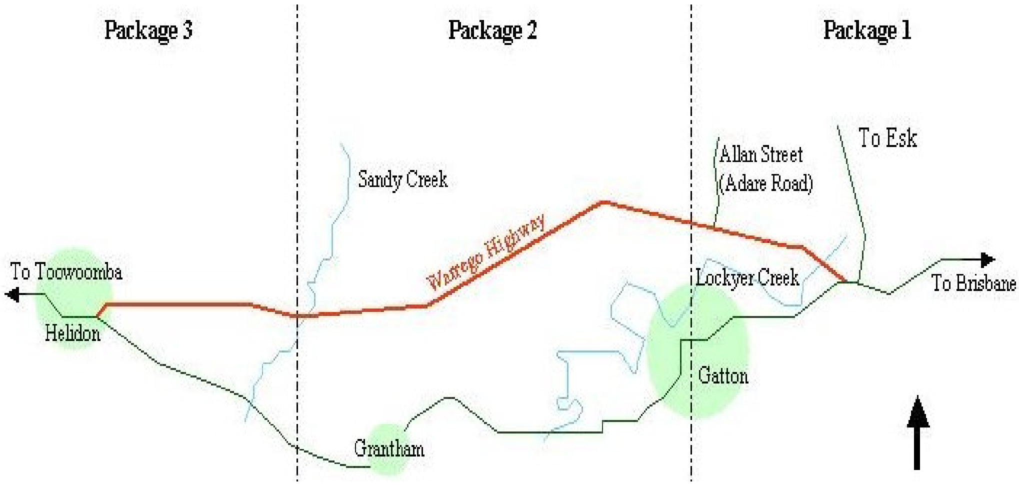


Figure 4.10 – Location of the Gatton Bypass and the construction packages for the duplication

Table 4.30 – Summary of major work carried out on the Gatton Bypass duplication in 2002-2003

Gatton Bypass Duplication Project: 114/18A/54		
Work	**Details**	**Cost**
Drainage	Culverts and associated work	\$3 Million
Bridges	Sandy Creek Bridge Lockyer Creek Bridge Gatton-Esk Interchange Helidon Interchange 3 other minor ones	\$9 Million
Earthworks	500,000m^3 excavation 400,000m^3 embankment	\$6 Million
Pavement	160,000m^3	\$9 Million
Surfacing	3000 tons of Asphalt 3000m^3 of aggregate 700 tons of bitumen	\$1 Million

The total cost of the duplication project was $47 million, with the project as a whole being completed around the start of 2004. Some of the work carried out is summarised in the table.

General Review of the Problem

The Gatton Bypass duplication was a major construction project and as such, considerable effort was put in by Main Roads personnel to ensure that the performance of the completed work was as good as possible.

Despite this, there have been some isolated pavement failures and deflection testing has revealed some possible weaknesses in the pavement structure. This report will examine the extent of these possible problems.

Scope of Work

Importance of the road

The Warrego Highway between Ipswich and Toowoomba (18A) is part of the National Highway system and an important road link for southeastern Queensland.

As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

The traffic volumes on this road are very high. In 2003, the AADT (Average Annual Daily Traffic) in the vicinity of the Bypass was about 10,000 vehicles/day (approximately half each way), including about 2,000 heavy vehicles, composing about 20% of the total traffic volumes. These heavy vehicles include trucks/buses, semi-trailers and articulated vehicles. Hence, the use of the road for the transport of freight is very important.

Magnitude of the failure

At present, apart from some minor isolated failures, the new pavement has performed fairly well. The focus of the investigation is more to examine the possible long-term performance.

Figure 4.11 – Gatton Bypass:

a) 'Rippling' of the pavement occurred in some sections, due to the paver that was laying the material being bumped by delivery vehicles

b) This rippling led to some isolated failures that had to be fixed

Figure 4.12 – Gatton Bypass:

a) Isolated ravelling of the asphalt surfacing just west of the Sandy Creek Bridge

b) Another view of the failures. These were corrected using patching.

Current risk to road users and risk of deterioration in the future

The new road is very safe to travel on, and the risk of deterioration in the near future is expected to be minimal.

Investigation Plan

The goal of the investigation was to assess the possible performance of the new pavement, using a visual investigation and deflection testing results.

4.4.2 Review Documents and Literature

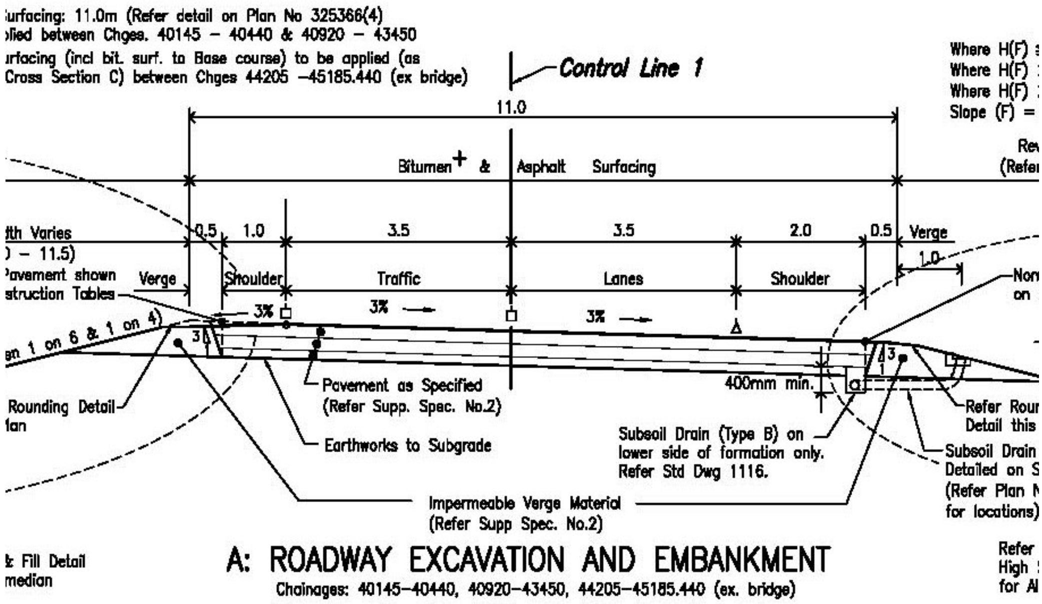
Plans

The plans for the job and a typical cross-section of the work were reviewed. This provided information about the work to be carried out and other information. The design life of the project was 20 years, with the design traffic loading being 1.8×10^7 ESA's. The assumed subgrade CBR for design was 10 (soaked) and the nominal road crossfall 3%. Each of the two lanes had a width of 3.5m, with the shoulders having a width of 1 - 2m.

The base was two 100 mm layers of type 1.1 unbound granular material. The subbase consisted of two 100 mm layers of type 1.2 material, and a 140 mm layer of type 2.5 material. This gave a total pavement depth of 540 mm, with the material being sourced from nearby quarries.

A somewhat unusual aspect of the construction was the choice of boxed construction. Impermeable verges were placed on either side of the pavement, the purpose of which was to prevent water entering the pavement. However, any water that got into the pavement structure would also possibly be unable to escape.

The concept of a verge to help prevent moisture entry into a pavement was adopted from Victorian practice, and the success of it in Queensland weather and rainfall conditions will only be able to be assessed in the future.



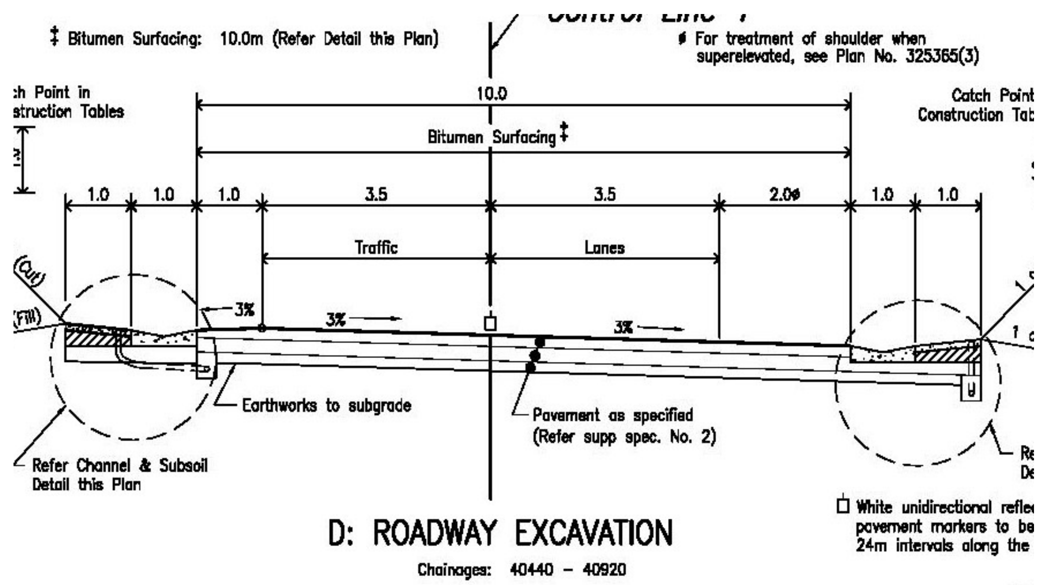


Figure 4.13 – Extracts showing typical pavement cross-sections for the Gatton Bypass

The verge material had to be able to meet stringent requirements, some of which were somewhat mutually exclusive. For example, low permeability was required, but so was low linear shrinkage and plasticity. While clays have low permeability, plasticity and shrinkage is normally high. Originally, it was intended that the verge material be sourced from material available on site. However, a shortfall in quantities required some to be sourced from off site.

Drainage Design

Major drainage structures for carrying water under the road, such as culverts, were aligned to correspond with those on the existing road. Subsoil drainage was also incorporated into the pavement structure, being located on both sides of the road, near the lower subbase level. The purpose of the subsoil drains was to enable the escape of any water that entered the pavement structure.

Pavement Materials Information

Type of material(s)

The pavement structure consisted of 2 x 100 mm layers of type 1.1 gravel as a base, 2 x 100 mm layers of type 1.2 gravel as an upper subbase, and a 140 mm layer of type 2.5 gravel as the lower subbase. Subgrade, sourced from material on site, consisted of CBR 10 material.

A supplementary specification was detailed for surface finish of the base material to ensure a hard surface with a reduced risk of aggregate embedment leading to bleeding. A three point pneumatic tyred roller was specified for the final base finish, as recent experience in the district indicated that these machines achieve a good tight surface, as they crush the exposed pavement material.

Generally, the surfacing along the project consisted of a two coat seal, the details of which are discussed below. However, a dense graded asphalt surfacing was used in some areas, such as over bridges. Overall, a sprayed seal surfacing was used along about 16.9 km of the job, with dense graded asphalt being used over about 3 km.

The prime consisted of AMC00 cutback bitumen. It was specified that it be left for at least 3 days before being trafficked by construction vehicles, otherwise a 5 mm covering would need to be applied.

While the specification stated that the primer should have been left for 10 days before sealing, it was decided to leave it for 3 weeks (21 days) to ensure that the kerosene cutter had evaporated, and the prime had had time to penetrate the base surface, to ensure a good bond between the seal and the pavement

The initial surfacing was designed to be a S2S (SOE) polymer modified binder with cutter applied, and using a lightly precoated 14 mm aggregate. It was important to ensure that the seal was back rolled using a pneumatic roller before any trafficking took place.

The second coat seal of 14 mm aggregate would be applied to help ensure a coarse surface texture and reduce the risk of moisture entering the pavement through the surface. This second seal would be completed by Main Roads some time after the first seal had been applied, and trafficking had taken place. The application rates would be decided based on how the initial seal performed and the surface texture of it.

Material test results

Materials were extensively tested before use in the project, due to the size of the work being carried out. During construction test results were generally satisfactory.

With both contractors, there were problems at one stage with subgrade and pavement materials being placed at a very low percentage of the optimum moisture content, with the density requirement being achieved by putting more compactive effort into the materials.

This occurred partly because of the perceived scarcity of water by the contractors, and also because the Main Roads Specifications were a bit ambiguous on this matter. It is not certain how this will influence the performance of the pavement.

4.4.3 Non-destructive Condition Survey

Visual Investigation

A visual investigation of the new pavement was conducted in July 2004, about 6 months after completion of the project. Apart from some minor patching work just west of the Sandy Creek Bridge caused by asphalt failures in January-February 2004, the road appeared to be in good condition, with low roughness and no noticeable rutting, cracking or other forms of failure.

PAVDEF Survey

A PAVDEF Deflection Survey was conducted along the new eastbound lanes of the Gatton Bypass in November 2003. The full data was available from the publication *PR 2254D PAVDEF Deflection Survey: Gatton Bypass Duplication* (Queensland Department of Main Roads, 2003). Graphs of the data are reproduced in Appendix F.

A brief overview of the results is provided in the following table. Only a summary of the data for the outer wheelpath of the outer lane and the inner wheelpath of the inner lane is shown, since this represents the extremes of the data.

These values can be examined to see how well they appear to satisfy the currently available guidelines, as shown below. It should be noted that the values used for the analysis had already been factored up or down to some extent (e.g. by using 90% highest or 10% lowest values), and this means that the results tend to be on the conservative side.

Table 4.31 – Summary of PAVDEF deflection testing data: Gatton Bypass eastbound

Parameter	10% Value	Median (50%)	90% Value
90% Highest Deflection (mm)	0.30 - 0.37	0.36 - 0.49	0.51 - 0.73
Mean Curvature (mm)	0.12 - 0.16	0.15 - 0.20	0.21 – 0.30
10% Lowest Subgrade CBR	11 – 15	19 – 23	24 – 25
10% Lowest Deflection Ratio	23 - 24	30	35 - 36

90% Highest Deflection (mm)

There are no currently available guidelines for this parameter, and it was found that this was strongly related to curvature. Therefore, this parameter will not be considered further.

Mean Curvature (mm)

The Pavement Rehabilitation Manual (Queensland Transport, 1992) states that values greater than 0.4 mm may suggest a lack of stiffness, a thin pavement or a cracked surface, while a value of less than 0.2 mm indicates a stiff pavement.

As can be seen from the above table, most of the data is below 0.20 mm indicating good stiffness, with 90% of the results were below 0.30mm, even for the outer wheelpath of the outer lane.

10% Lowest Subgrade CBR

The design subgrade CBR for the project was 10. As indicated in the above table, this value appears to have been exceeded by a wide margin along most of the job.

10% Lowest Deflection Ratio

Baran (1994) suggests the following guidelines for deflection ratio using the PAVDEF Deflection Testing device, expressed as a percentage:

>60	Representative of a bound base
40 – 50	Unbound Granular Base
< 40	Could represent a weakness in the base layer

As can be seen from the above table, the deflection ratio appears to be quite low, considering the above criteria. However, since the 10% lowest values were used for the analysis, there is some conservatism in the results, and so they could be considered acceptable, especially since there is no really precise relationship between the value of this parameter and the pavement integrity.

Falling Weight Deflectometer (FWD) Survey

As a result of the results obtained from the PAVDEF Survey, a Falling Weight Deflectometer (FWD) survey was carried out in November and December 2003, to give further information for analysis.

The survey was conducted at 19 test sites, over a total length of about 3475m (17% of the length of the job), with loads of 40kN and 60kN being used at all locations. The results were summarised in a report, *Gatton Bypass Duplication - Analysis of FWD Deflection Results,* prepared in February 2004, by the Pavements, Materials and Geotechnical Division, Road Systems and Engineering Group, Queensland Department of Main Roads.

Review of Deflection Criteria

In this report, the origin of the deflection ratio criteria for deflection testing were reviewed, and it was concluded that due to the different testing devices used and the limited research in this area, that interpretation of this information is subjective, and at best can only give an indication of the likely pavement stiffness.

Estimation of Pavement and Subgrade Moduli from FWD Deflection Results

The granular material and subgrade moduli were back calculated from the FWD deflections using a computer program. It was necessary to make assumptions regarding the pavement composition. Graphs of the data are reproduced in Appendix G. The results of the analysis was summarised in the report, including the following main points:

- The estimated in-situ moduli of the subgrade exceeded the design value at more than 95% of the test sites.

- The estimated in-situ moduli of the top 133 mm of the granular base exceeded the design modulus at more than 95% of the sites.

- For the other granular layers, the design moduli were generally achieved (at more than half the sites), but at some sites, the estimated in-situ moduli were below the design value.

- The area of most concern in terms of future performance is from job chainage 31800 – 31980 on both lanes (road chainage 62.16 – 62.34 km). The deflections were high and the estimated granular moduli were at or below the design values.

It was noted that there is considerable uncertainty when back calculating layer moduli, since there is more than one combination of layer moduli where the predicted deflection bowls match the measured values. Often a low modulus for one layer occurs at the same time as for a high modulus for another layer. Therefore, the estimated values should be used with some caution.

4.4.4 Conclusion

A visual inspection of the site in July 2004 showed the road section to be in good condition, with no noticeable signs of failure, apart from some patching just west of Sandy Creek.

A PAVDEF Deflection Survey carried out along the length of the job in November 2003 showed that there were some doubts about the stiffness of the pavement. While the curvature function showed the pavement to be in good condition, the deflection ratio was quite low, indicating a possible weakness. However, due to the difficulty of interpreting these results, this is open to question.

A Falling Weight Deflectometer (FWD) survey was carried out in November and December 2003 to further study the performance of the pavement. Layer moduli were calculated for both the pavement layers and the subgrade. This revealed that the estimated subgrade moduli exceeded the design value at most sites, as did that for the top 133 mm of the pavement.

For the rest of the pavement layers, the design moduli was exceeded at more than half the sites, with some sites having an estimated moduli less than the design value. The worst location was from chainage 62.16 – 62.34 km. This section is soon after the beginning of package 2, and was one of the first sections where the pavement was placed.

Therefore, it is possible that the low estimated layer moduli in this area was due to slightly inferior construction methods that were improved as construction continued and pavement was placed in other areas.

In conclusion, while initial deflection testing indicated possible weaknesses in the pavement structure, more detailed study including back calculation of layer moduli tended to suggest that the pavement layers had stiffness exceeding the design values. The visual investigation also revealed few signs of failure.

4.5 Tabletop Turnoff

4.5.1 Plan the Investigation

General Review of the Problem

The project 149/18A/24 was constructed on the Warrego Highway between Toowoomba and Ipswich, taking place on a section of the Toowoomba range, between chainages 87.71 and 88.75 km, a length of about 1.04 km. The project was completed in October 2002.

The main work of the project involved the construction of a concrete barrier on the median to separate the up and down traffic lanes, and the upgrade of the intersection at the Tabletop Road Turnoff.

The project also involved an asphalt overlay of the down (eastbound) traffic lanes between chainages 87.79 and 88.54 (a length of 0.75 km), and pavement reconstruction and sealing of the up (westbound) traffic lanes between chainages 87.71 and 88.74 (a length of 1.03 km).

The eastbound (down) lanes in the vicinity of the project have experienced very severe rutting in the wheelpaths over a length of about 100-200m, mostly in the outer lane, where the majority of the heavy vehicles travel.

Scope of Work

Importance of the road

The Warrego Highway between Ipswich and Toowoomba (18A) is part of the National Highway system and an important road link for southeastern Queensland. As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

Figure 4.14 – Tabletop Turnoff:

a) A view showing the location where rutting is occurring

b) A closer view of the rutting that is occurring in the outer wheelpath of the eastbound outer lane, as well as a lack of surface texture

The traffic volumes on this road are very high. In 2003, the AADT (Average Annual Daily Traffic) in the vicinity of the project was about 18,200 vehicles/day (approximately half each way), including about 2,300 heavy vehicles, composing about 12.5% of the total traffic volumes. These heavy vehicles include trucks/buses, semi-trailers and articulated vehicles. Hence, the use of the road for the transport of freight is very important.

Magnitude of the failure

The rutting being experienced in the affected section is very high, with the rutting under a 30cm straightedge being about 20-30mm. It is quite noticeable, and vehicles changing lanes or direction in the affected area are destabilised when moving across the ruts.

A review was made of the asset management data for the eastbound lanes of the Warrego Highway (18A) to assess whether the failure was occurring greatly enough to be detected at the network level, or whether the failure was more isolated.

The data is averaged over 1 km intervals, and so the data points Ch 87 - 88 km and 88 - 89 km contain the affected section. The results are shown in the following table.

Table 4.32 – Roughness and rutting near the Tabletop Turnoff, compared to the road average

Condition	Average Ch. 29 – 92 (Eastbound)	Ch. 87 – 89	% of Road Average
Roughness (count)	62	59	95
Rutting (mm)	4.9	3.8	78

As can be seen from the above table, the data over the affected section does not indicate that the failure is taking place, and this would be expected, considering that the failure is of limited extent.

Current risk to road users

There is some risk posed by the ruts, due to the magnitude of them, and the effect of this on vehicles moving through the affected area.

Risk of further deterioration in the future

It is possible that the ruts will continue to worsen, although this is dependant on what the primary cause(s) of the failure is. While the ruts may stabilise with no further increase in depth, they should still be rehabilitated to present a uniform road surface for the safety of road users.

Investigation Plan

The purpose of the investigation was to investigate possible failure causes, and suggest possible rehabilitation treatments for the affected pavement.

4.5.2 Review Documents and Literature

Plans and Pavement History

Since the rutting is confined to the eastbound (down) section of the road, only information about this side of the road will be discussed. The project 149/18A/24 involved the placement of a 40 mm overlay between chainages 87.79 and 88.54 (a length of 0.75 km).

The current pavement layers along this section of road is summarised in the table. The information was taken from the ARMIS system. As can be seen from the information, in the vicinity of the failure, the pavement layers are quite old, apart from the asphalt that was recently placed.

Table 4.33 – Pavement details for eastbound lanes near Tabletop Turnoff

Ch. Range (km)		Length (km)	Date	Material Type
87.792	87.83	0.038	Feb-04 Oct-02 Jan-81 Jan-81	12 mm PMB Spray Seal 40 mm Dense Graded Asphalt 20 mm Spray Seal 150 mm Granular Material
87.83	88.25	0.42	Oct-02 Jan-81 Jan-81	40 mm Dense Graded Asphalt 20 mm Spray Seal 150 mm Granular Material
88.25	88.43	0.18	Oct-02 Jan-91 Jan-91	40 mm Dense Graded Asphalt 20 mm Spray Seal 150 mm Granular Material
88.43	88.542	0.112	Oct-02 Jan-83 Jan-83	40 mm Dense Graded Asphalt 20 mm Spray Seal 150 mm Granular Material

4.5.3 Non-destructive Condition Survey

Visual Examination

A visual examination of the failure was carried out in July 2004, and the results of this will be discussed below.

The ruts are quite deep and any vehicles changing lanes across the ruts are destabilised by them. This means that rehabilitation work would probably be required in the future, even if the depth of rutting remains at its current level.

The following was observed about the location of the rutting:

- It was occurring almost completely in the outer lane, predominantly in the outer wheelpath.

- It was only occurring on the eastbound lanes, where the surfacing was asphalt.

- Near the rutting where new asphalt had not been placed, the older surfacing was exhibiting signs of structural or fatigue cracking, as indicative of an old surfacing or pavement age.

Drainage and Moisture

Due to the steep grade of the road in the vicinity of the failure, water would be unlikely to pond nearby, and permeate into the pavement.

It was observed that on the right-hand (southern side), the water could easily drain down the road surface, whereas on the left-hand (northern) side, the road was constructed in a cutting, and while moisture could possibly infiltrate into the pavement, there was no sign of this.

Topography and Alignment

The grade in the vicinity of the failure is quite steep downwards, and it is possible that the braking forces of heavy vehicles could be contributing to the failure. The alignment is straight, and at a bend slightly further on, there is no sign of failure.

Pavement Lateral Profiles

The use of a 30cm straightedge to measure the rutting confirmed the rather large magnitude of failure, with the rutting under the straightedge being about 20-30mm. This magnitude means rehabilitation will probably required in the future.

Figure 4.15 – Tabletop Turnoff:

a) Structural cracking in an area nearby where no asphalt had been placed

b) A closer view of the magnitude of rutting that is occurring in the outer wheelpath of the eastbound outer lane, as well as a lack of surface texture

4.5.4 Determine Probable Cause(s) of Failure

Determine Information in support/refusal of each Failure Hypotheses

Since the type of failure is rutting, the possible causes of this will be examined, with the results shown in the following table.

Table 4.34 – Possible causes of rutting and associated information

Possible Failure Cause	Information
Inadequate pavement thickness	Possibly, since there is only 150 mm of granular material. However, this does not explain why failure has only occurred where asphalt was recently placed.
Weak subgrade	Possibly may be weak subgrade under outer pavement due to moisture entry.
Weak base	Pavement granular material is about 10-20 years old, and may be deteriorating due to age.
Surfacing lack of strength / stability	Rutting where new asphalt was placed indicates that the failure may be occurring in this material.
Inadequate Compaction	No evidence.
Poor Material Quality	No evidence.
Excessive Moisture	Possibly the cause of material weakness.
Very high traffic loading (> 10^7 ESAs)	Traffic volumes and loading are very high and it is possible that the asphalt mix could not cope, without undergoing rutting.
Inappropriate mix design (asphalt)	No evidence.
Interaction between layers (e.g. excess cutter moving into next layer)?	Unlikely, since previous surfacing was very old.

Determine Probable Cause(s) of Failure

From examining the above, it can be seen that based on the limited investigation done already, possible failure causes are as follows:

- Inadequate pavement thickness
- Weak subgrade, base, or asphalt surfacing (or a combination)
- Very high traffic loadings

Since the failure has occurred mostly where new asphalt was placed, it is likely that the cause of the rutting is related to the asphalt, although all of the above factors may possibly be contributing to the failure.

The above conclusions could be confirmed as required in the future, by carrying out some materials testing.

4.5.5 Determine Best Rehabilitation Treatment

Since the information known about the cause of the failure at the present time is rather limited, it is difficult to perform a detailed examination of possible rehabilitation options. However, several possible options are as follows:

- Correct or mill surface, then apply an asphalt or bitumen surfacing
- Reconstruct pavement or overlay
- Improve moisture control
- In-situ stabilisation

Since it appears that the failure is related to the asphalt, the rehabilitation treatment should aim to correct this deficiency, while accepting that future investigation in the future may reveal an alternative failure cause. A new dense graded asphalt surfacing may perform satisfactorily. However, the use of a newer treatment, such as stone mastic asphalt with a greater resistance to rutting may prove to be a better option in the long run.

4.6 Warrego Highway through Toowoomba

4.6.1 Plan the Investigation

General Review of the Problem

The Warrego Highway through Toowoomba consists of about 11 km of road, all of which has two lanes in two directions. The section consists of about 3 km of 18A (Ipswich-Toowoomba) and 8 km of 18B (Toowoomba-Dalby), as shown in the figure below.

Generally, many parts of the road section are showing some signs of failure or fatigue. However, due to the many varied treatments applied along the length over the years, it is somewhat difficult to determine a single treatment strategy that maintains the section to the highest possible standard at the lowest cost.

Figure 4.16 – State-Controlled Roads through Toowoomba

Table 4.35 – Traffic volumes on the Warrego Highway through Toowoomba: 2003, both ways

		Traffic Volumes			
Ch. (km)	**Details**	**Total**	**Light**	**Heavy**	**%**
18A – Warrego Highway (Ipswich – Toowoomba)					
92	92.0 – Beginning of Cohoe St 92.3 – Intersection with Herries St 92.7 - Beginning of James St	18,650	16,440	2,210	11.9
93	93.2 - Intersection with MacKenzie St 93.7 - Intersection with Kitchener St	20,010	18,010	2,000	10
94	94.6 - Intersection with Hume St (New England / 22A)	20,420	18,280	2,140	10.5
18B – Warrego Highway (Toowoomba – Dalby)					
0	0.0 - Intersection with 18A and Ruthven St (New England / 22B)	23,580	21,340	2,240	9.5
1	1.0 - Intersection with West St	21,540	19,390	2,150	10
2	2.2 - Intersection with Anzac Avenue (Gore Highway / 28A)	17,790	15,780	2,010	11.4
3	3.7 - Intersection with Taylor St (Toowoomba-Cecil Plains Rd / 324)	15,550	13,940	1,610	10.4
4	4.5 - Beginning of Bridge St	13,830	13,210	620	4.5
5	5.3 - Intersection with Richmond Dr	15,800	15,040	760	4.9
6	6.6 - Intersection with MacDougall St	13,970	13,180	790	5.7
7	7.3 - Intersection with Boundary Rd	10,860	10,020	840	7.8
8	8.1 - End of Two Lanes each way, Intersection with Nugent Pinch Rd	10,860	10,040	820	7.6

Scope of Work

Importance of the Road

The Warrego Highway between Ipswich and Toowoomba (18A) is part of the National Highway system and an important road link for southeastern Queensland. As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

The traffic volumes on the Warrego Highway through Toowoomba are very high, and consist of both town commuters and heavy vehicles, including road trains and B-Doubles at the western end. The New England and Gore Highways intersect with the Warrego Highway at particular points, adding to the traffic volumes, which are shown in the table, as at 2003, averaged at 1 km intervals.

Magnitude of the Failure

A review was made of the asset management data for the Warrego Highway (18A and 18B) within the Toowoomba city limits to assess whether the magnitude of the roughness and rutting that is occurring is significantly greater than other sections of State-Controlled road within the boundaries of Toowoomba. The results of this are shown in the table.

As can be seen from this table, despite (or perhaps because of) the greater traffic volumes on 18A and 18B, the roughness and average rutting on these roads within Toowoomba is significantly greater than that on the other roads. It can be seen that this does not appear to be due to the seal or pavement ages being significantly greater than those for the other roads.

Current Risk to Road Users

The speed limit on the Warrego Highway through Toowoomba is predominantly 60 km/h. This means that pavement deformation is not likely to be as much a cause of accidents as similar failures outside urban areas, where the speed limit is 100 km/h or greater.

However, it is important to take into consideration the much greater traffic volumes that use the roads within Toowoomba, compared to outside the urban area. This means that it would be desirable to still provide as high a level of service as is possible.

Table 4.36 – Average asset management data for state controlled roads through Toowoomba

		Roughness	Rutting	AADT	Age (years)	
Road	**Ch. (km)**	**Austroads Counts**	**Average (mm)**	**Vehicles per day**	**Seal**	**Pavement**
18A	92-95	81	6.4	20,020	7	20.8
18B	0-8	67	4.9	16,620	7.1	20.6
22A	115-118	73	2.6	15,400	13.8	23.5
22B	0-5	58	3.4	15,430	10.6	24.7
28A	0-4	55	4.3	12,980	5.9	19
324	0-3	43	3.6	10,670	7.3	11.6

Another factor in urban areas is the potential loss of skid resistance, particularly near intersections and other locations where heavy braking may be required. Some sections of the Warrego Highway through Toowoomba are experiencing bleeding and flushing, leading to a possible reduction in skid resistance at these locations, and this problem may need to be examined further in the future.

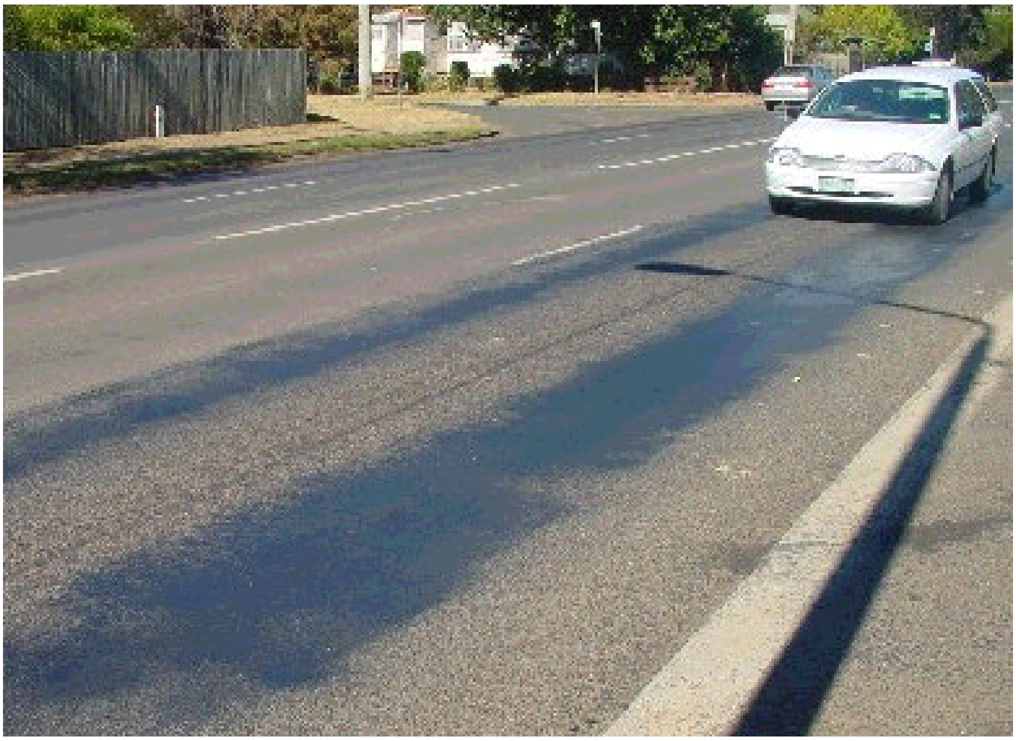


Figure 4.17 – Warrego Highway through Toowoomba:

a) Bleeding on Bridge Street west of Boundary Road

b) Bleeding on Taylor Street, north of the intersection with Hursley Road

Overall, the section of the Warrego Highway through Toowoomba is still quite safe to drive, and the overall influence of the pavement condition on the safety of road users is fairly minor compared to other factors such as intersections and heavy traffic congestion.

Investigation Plan

The purpose was to investigate the Warrego Highway through Toowoomba to determine the need for rehabilitation work, and what the best possible treatments may be. Due to the wide variety of treatments applied along the length of the road, only some specific sections will be investigated.

4.6.2 Review Documents and Literature

Pavement History

From the ARMIS system and plans of previous jobs, the pavement history was reviewed. A summary of the asset management data as at mid-2004 is provided in the table. If there are two values shown, there were measurements taken for both the westbound and eastbound lanes, in that order.

Pavement Materials Information

From the ARMIS system, it was possible to review the materials used in the road throughout Toowoomba, and the results are summarised in the following tables. The lengths are greater than the 11 km previously stated, since multiple carriageways (one eastbound, one westbound) were considered, from 18A: Ch. 92.18 to 18B: Ch. 8.1 km, giving a total length of about 16.05 km.

The table gives an indication of what surfacing are used on the roads. From the ARMIS data, it is possible to select the topmost layers only, and sum the lengths for each particular type.

The next table gives an indication of what an 'average' pavement composition is. However, it should be noted that no road section has this actual pavement structure. This is similar to how the average of a list of numbers does not need to equal one of the numbers.

From the ARMIS data, for each material layer recorded, the m^2/m width of road of a certain material type was found by summing the length of each layer by the depth. This was then converted into an 'average' depth by dividing this value by the total length of road, which in this case was 16.05 km.

Table 4.37 – Asset management data for the Warrego Highway through Toowoomba

Road	Chainage (km)	Age (years)		Roughness	Rutting
		Seal	Pavement	Austroads Counts	Average (mm)
18A	92	6.3	18.5	85	5.3
	93	7.4	21.8	75	6.5
	94	7.2	22.2	84	7.4
	95	19.5	19.5	79	10.9
18B	0	8.5	25.5	85	4.3
	1	6.7	--	68, 89	5.9, 7.1
	2	8.1	--	67, 79	5.6, 4.3
	3	6.8	34.5	93	8.5
	4	3	34.5	48	4.4
	5	9.4	27.9	57, 57	4.1, 2.3
	6	6.5	8.9	74, 51	6.7, 3.7
	7	6.5	8.9	55, 52	3.7, 3.1
	8	9.7	12.7	61	4.7

Table 4.38 – Surfacing used on Warrego Highway through Toowoomba

Surfacing Type	Length (km)	% Of Total
Dense Graded Asphalt	9.97	62
Spray Seal	5.70	36
Slurry	0.38	2
Total	**16.05**	**100**

Table 4.39 – An 'average' pavement composition for the Warrego Highway through Toowoomba

Material Type	m^2/m width of road	Average depth (mm)
Dense Graded Asphalt	880	55
Spray Seal	319	20
Slurry Surfacing	1.0	0.1
Granular Pavement	3500	218
Total	**4700**	**293**

4.6.3 Non-Destructive Condition Survey

Visual Investigation

Due to the wide variety of sections along this length of road, a summary of the main points noted about some specific sections will be listed.

18A: Ch. 92.3 – 92.76 – Cohoe St, from Herries St to James St

The pavement is rutting in both the east and westbound outer lanes. This rutting had been corrected in some areas with grader placed asphalt.

18A: Ch. 92.76 – 93.76 – James St, from Cohoe St to Kitchener St

The existing asphalt overlay along much of this section was placed in 1989, with rejuvenation treatments being carried out in 1993. The pavement is generally fatigued with rutting and cracking, with the outer lanes being worst.

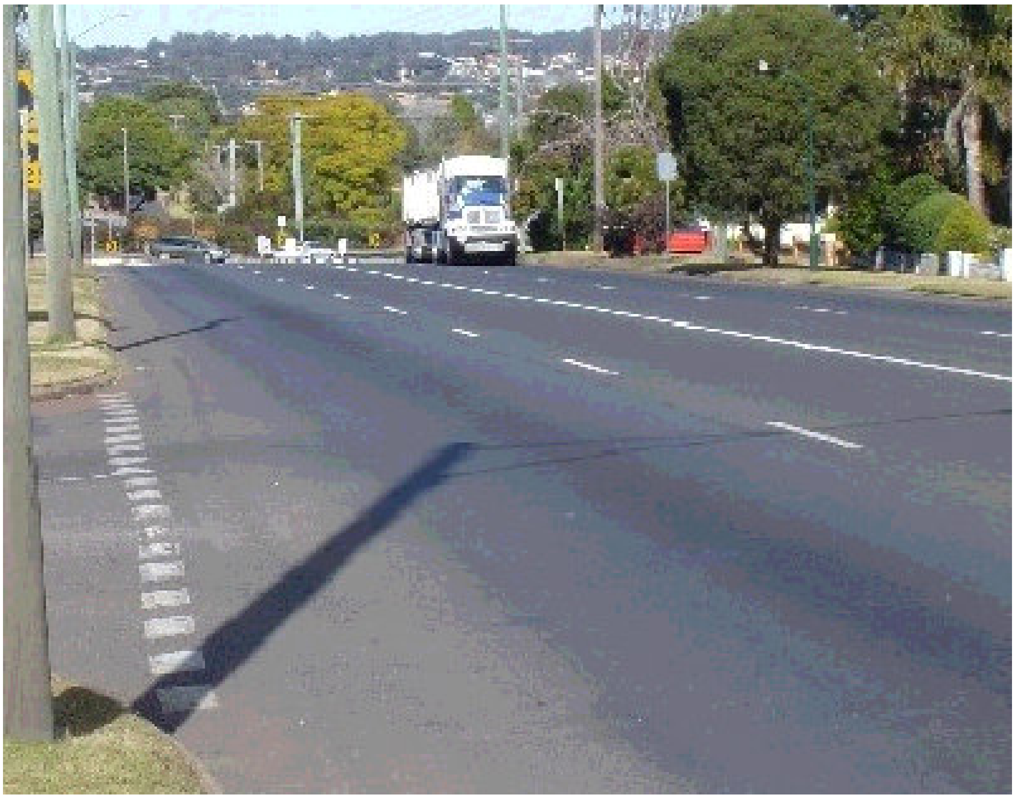


Figure 4.18 – Warrego Highway 18A, along Cohoe Street:

a) Looking south towards the intersection with James Street

b) Looking north, showing some asphalt placed to correct rutting in the outer lane

Figure 4.19 – Warrego Highway 18A, along James Street near Kitchener Street:
a) Structural cracking in the outer westbound lane
b) View looking west towards Kitchener Street intersection

The section from Cohoe to MacKenzie (92.76-93.27) was rehabilitated in the May 2004. The rehabilitation consisted of a 12 mm S2S PMB seal, overlaid with 50 mm of DG14 polymer modified asphalt.

The section from MacKenzie St to John St (93.27-93.6) has asphalt patches (about 3-5 years) old fatigue cracking in the outer lanes in both directions. There was also some cracking adjacent to trees. It was observed that there was very little rutting, with the main type of failure being fatigue cracking.

18A: Ch. 93.95 – 94.58 – James St, from Macarthur St to Hume St

An asphalt overlay was placed over this section in 1977, and since then some other overlays and rejuvenation treatments have been applied. There is rutting, cracking and signs of pavement failure. The outer lanes are the worst, with eastbound being especially bad.

18B: Ch. 2.6 – 3.74 – Tor St, from Rob St to Taylor St

In the early 1980s this section was upgraded from a two lane, two way road to a four lane road by including the existing parking lanes. Only a thin layer of asphalt was applied to these parking lanes before they became the outer lanes of the widened section. A reseal of the section was done in 1983, and an asphalt overlay occurred in 1995-1996.

In the outer lanes there is rutting and fatigue cracking. Fairly recently, a 10 mm PMB spray seal had been placed over badly fatigued areas and appeared to be performing well despite the road condition and the traffic volumes.

18B: Ch. 5.8 – 7.9 – Bridge St, from Greenwattle St to Nugent Pinch Rd

The westbound section was constructed in 1995. Major bleeding occurred in the wheelpaths soon after opening, due to the placement of the seal in winter followed by an early hot summer. The seal was overlaid with Novachip in November 1998, but by 1998 the excess bitumen had worked its way to the surface. Some small aggregate was applied to stop stickiness, and this had some short-term success.

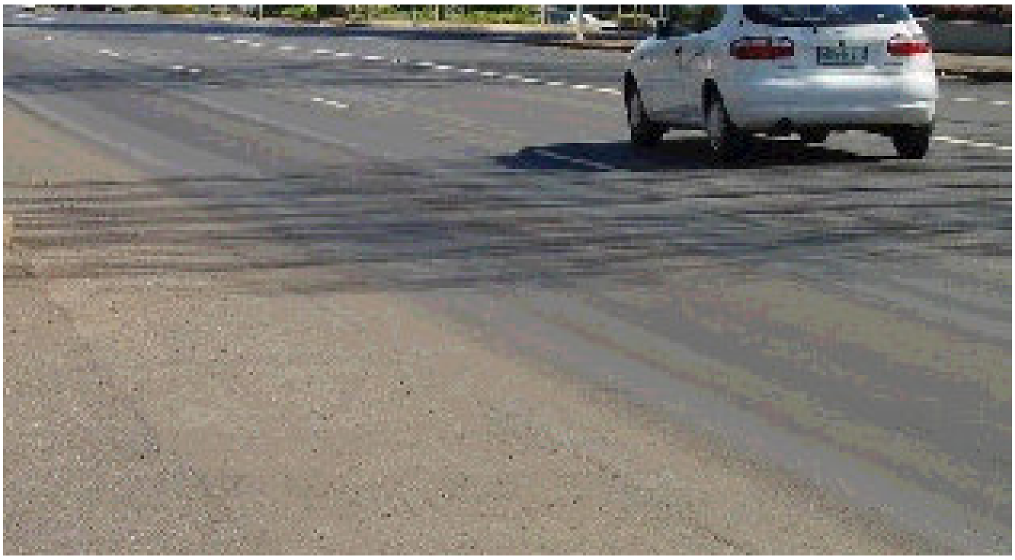


Figure 4.20 – Warrego Highway 18A, along James Street near Hume Street:

a) Looking west

b) Looking east, showing rutting in outer westbound lane

Figure 4.21 – Warrego Highway 18B, along Tor Street north of Taylor Street:

a) Typical fatigue cracking in the outer lane

b) Bleeding occurring in the outer northbound lane

Figure 4.22 – Warrego Highway 18B, along Bridge Street:

a) Bleeding and roughness in westbound lanes leading to MacDougall Street

b) Bleeding and rutting in westbound lanes leading to MacDougall Street

In 1999, HIPAR (Hot in Place Asphalt Recycling) was used on the westbound lanes from Greenwattle St to Boundary Rd to attempt to soften the bitumen enough to allow new aggregate to stick to it. Only part of the section was completed, due to the fumes generated.

Safety is a concern in this section, due to the wheelpaths of both outer lanes and much of the inner lanes being badly flushed, and with significant rutting in the outer wheelpaths of the outer lane, mostly between Greenwattle St and Boundary Rd.

PAVDEF Survey

A PAVDEF Deflection Survey was conducted along all lanes, both eastbound and westbound, of the Warrego Highway through Toowoomba in May 2004. The chainages tested were 18A: 92.32 – 95.0 km, and 18B: 0.0 – 8.10 km.

The full data was recorded in *PR 2331A PAVDEF Deflection Survey: Warrego Highway through Toowoomba* (Queensland Department of Main Roads, 2004b), with graphs being provided in Appendix H.

A brief overview of the results is provided in the tables. Only a summary of the data for the outer wheelpath of the outer lane and the inner wheelpath of the inner lane for each direction is shown, since this represents the extremes of the data.

These values can be examined to see how well they appear to satisfy the currently available guidelines, as discussed below. It should be noted that the values used for the analysis had already been factored up or down to some extent (e.g. by using 90% highest or 10% lowest values), and this means that the results tend to be on the conservative side.

Table 4.40 – Summary of PAVDEF deflection testing data: Warrego Highway through Toowoomba – Westbound

Parameter	10% Value	Median (50%)	90% Value
90% Highest Deflection (mm)	0.29 – 0.34	0.53 – 0.64	0.71 – 1.00
Mean Curvature (mm)	0.06 – 0.07	0.13 – 0.14	0.23 – 0.27
10% Lowest Subgrade CBR	5 - 7	9 - 12	17 – 22
10% Lowest Deflection Ratio	27 - 29	41 - 46	53 – 54

Table 4.41 – Summary of PAVDEF deflection testing data: Warrego Highway through Toowoomba – Eastbound

Parameter	10% Value	Median (50%)	90% Value
90% Highest Deflection (mm)	0.32 – 0.35	0.53 – 0.66	0.76 – 0.99
Mean Curvature (mm)	0.07 – 0.08	0.12 – 0.16	0.24 – 0.25
10% Lowest Subgrade CBR	4 - 7	8 - 10	14 – 18
10% Lowest Deflection Ratio	31 - 32	41 - 44	51 – 52

90% Highest Deflection (mm)

There are no currently available guidelines for this parameter, and it was found that this was strongly related to curvature. Therefore, this parameter will not be considered further.

Mean Curvature (mm)

The Pavement Rehabilitation Manual (Queensland Transport, 1992) states that values greater than 0.4 mm may suggest a lack of stiffness, a thin pavement or a cracked surface, while a value of less than 0.2 mm indicates a stiff pavement.

As can be seen from the above tables, most of the data is below 0.20 mm indicating good stiffness, with 90% of the results were below 0.30mm, even for the outer wheelpath of the outer lanes. Overall, the results for either direction were quite similar.

10% Lowest Subgrade CBR

As indicated in the above tables, median subgrade CBR is about 8-10, while the value for both directions being similar.

10% Lowest Deflection Ratio

Baran (1994) suggests the following guidelines for deflection ratio using the PAVDEF Deflection Testing device, expressed as a percentage:

>60	Representative of a bound base
40 – 50	Unbound Granular Base
< 40	Could represent a weakness in the base layer

As can be seen from the above table, the deflection ratio appears to be satisfactory, with the median value being about 40 – 44, considering the above criteria. It should be noted that there is some conservatism in the results, and that there is no really precise relationship between the value of this parameter and the pavement integrity.

4.6.4 Determine Best Rehabilitation Treatment

Due to the wide variety of pavement types and depths in the road section, this section will only look at some possible rehabilitation treatments for specific sections.

18A: Ch. 93.95 – 94.58 – James St, from Macarthur St to Hume St

Since the outer lanes are worst, rehabilitation and strengthening of the outer lanes would be probably be required, prior to an asphalt overlay full width incorporating rut resistant asphalt.

18B: Ch. 2.6 – 3.74 – Tor St, from Rob St to Taylor St

Since the outer lanes are worst, rehabilitation and strengthening of about 50% of the outer lanes would be probably be required, prior to an asphalt overlay full width incorporating rut resistant asphalt.

18B: Ch. 5.8 – 7.9 – Bridge St, from Greenwattle St to Nugent Pinch Rd

The first step in the rehabilitation would be to mill off the existing surface for the full length of both outer lanes and about 1000m of the westbound inner lane to remove the worst of the bleeding and flushing.

The pavement is failing for about 1640m of the westbound lanes and 840m of the eastbound lanes. The problem is probably in the base material, so an in-situ rehabilitation treatment such as recycling or stabilisation could be used to treat a total depth of 200mm: 150 mm of base and 50 mm of subbase. These areas could be sealed with a fabric seal, followed by a thin layer of Novachip over the whole area.

Table 4.42 - Cost calculations for James St (Macarthur-Hume)

Item	Amount	Details
Unit Costs		
Milling off material ($/m^2)	8	Removal of material for strengthening
Disposal of material ($/m^3)	11	Disposal of excavated material
Asphalt Cost ($/m^3)	336	Density = 2.4 ton/m^3 Cost = $140/ton
Quantities		
Length (m)	630	
Width (m)	15	Pavement width
Area for overlay (m^2)	9,450	Length x Width
Area needing prior strengthening (m^2)	5,040	Both outer lanes over full length – 4m width
Volume of material to be replaced (m^3)	756	Area needing strengthening x 150 mm depth
Thickness of asphalt overlay (m)	0.03	30 mm overlay of DG10
Costs ($)		
Milling off	40,500	Milling Cost x Area needing strengthening
Disposal	8,500	Disposal Cost x Volume of material to be replaced
Asphalt for restrengthening	254,250	Asphalt Cost x Volume of material to be replaced
Asphalt overlay	95,500	Asphalt Cost x Thickness of Overlay x Area of Overlay
Contract Cost	398,000	Total of milling, disposal, asphalt for restrengthening and overlay
Project Cost	454,000	Plus 14% for Overhead and supervision

Table 4.43 – Cost calculations for Tor St (Rob-Taylor)

Item	Amount	Details
Unit Costs		
Milling off material ($/m^2)	8	Removal of material for strengthening
Disposal of material ($/m^3)	11	Disposal of excavated material
Asphalt Cost ($/m^3)	336	Density = 2.4 ton/m^3 Cost = $140/ton
Quantities		
Length (m)	1,125	
Width (m)	14	Pavement width
Area for overlay (m^2)	15,750	Length x Width
Area needing prior strengthening (m^2)	3,940	50% of total outer lanes - 3.5m width
Volume of material to be replaced (m^3)	394	Area needing strengthening x 100 mm depth
Thickness of asphalt overlay (m)	0.03	30 mm overlay of DG10
Costs ($)		
Milling off	31,750	Milling Cost x Area needing strengthening
Disposal	4,500	Disposal Cost x Volume of material to be replaced
Asphalt for restrengthening	132,500	Asphalt Cost x Volume of material to be replaced
Asphalt overlay	159,000	Asphalt Cost x Thickness of Overlay x Area of Overlay
Contract Cost	327,000	Total of milling, disposal, asphalt for restrengthening and overlay
Project Cost	373,000	Plus 14% for Overhead and supervision

Table 4.44 - Cost calculations for Bridge St (Greenwattle – Nugent Pinch)

Item	Amount	Details
Unit Costs		
Milling ($/m^2)	6	Cost of milling over areas with worst bleeding
In-situ rehabilitation ($/m^2)	18	Cost of stabilisation or recycling
Fabric sealing ($/m^2)	6	Cost of sealing rehabilitated areas
Novachip ($/m^2)	9	Cost of Novachip resurfacing over milled areas
Quantities (m)		
Length (m)	2840	Total length
Width (m)	8.7	Average, taking into account varying needs on both WB and EB
Area for milling and resurfacing (m^2)	24716	Length x Width
Area needing in-situ rehabilitation and fabric sealing	6000	Length of Failures in both directions (1640+840) x 2.4m lane width
Costs ($)		
Milling	148,500	Milling cost x Area
In-situ rehabilitation	108,000	Rehabilitation cost x Area
Fabric sealing	36,000	Fabric Sealing cost x Area
Novachip	222,500	Novachip cost x Area
Contract Cost	515,000	Total of milling, rehabilitation, sealing and Novachip
Project Cost	587,000	Plus 14% for Overhead and supervision

4.7 Yaralla Deviation

4.7.1 Plan the Investigation

General Review of the Problem

The Yaralla Deviation project, 124/18C/16, was a project constructed 10 km west of Dalby on the Warrego Highway (18C), with the project being completed in January 2002. The length of the job was 18.6 km, from chainage 5.04 – 23.66 km, and it was constructed as an open tender contract by the company Stockport at a cost of about $13 million.

The purpose of the project was to construct a deviation of the Warrego Highway to bypass sections of flood-prone highway by transferring traffic to the Yaralla Road that ran parallel to the existing highway. This was necessary due to floodwaters frequently closing off sections of the highway during periods of heavy rainfall.

The alterations involved the upgrading of the Yaralla Road to an 11m wide two-lane, two-way road with a number of drainage crossings and intersections. The work for the project included three stages. The first stage was the full construction of a new three-kilometre deviation to link up with Yaralla Road, followed by an upgrade of a two-kilometre section of the existing highway. The final stage included the full upgrading of the Yaralla Road, which was made up of both gravel and bitumen surfacing, into a fully sealed fifteen-kilometre straight section of highway.

Since the soil in the vicinity of the project was known to be a black soil with high shrink and swell characteristics, in-situ stabilisation of the subgrade with quicklime was carried out to try and mitigate these effects.

Since the completion of the project, there had been signs of failure along the length of the project, including bleeding of the surfacing and some pavement deformation of the outer wheelpaths, including rutting that allows water ponding in the outer wheelpaths. If these failures continue to worsen, rehabilitation may be necessary in the future.

Figure 4.23– Yaralla Deviation:

a) A view of a failure in the outer wheelpath

b) A view of two isolated failures

Scope of Work

Importance of the road

The Warrego Highway between Dalby and Miles (18C) is part of the National Highway system and an important road link for the Darling Downs. As part of the National Highway system, funding is provided by the Federal Government, with all work being administered by the Queensland Department of Main Roads on their behalf.

The traffic volumes on this road are fairly high for a rural road. In 2003, the AADT (Average Annual Daily Traffic) in the area of the job was about 1710 vehicles/day, including about 270 heavy vehicles, composing about 16% of the total traffic volumes. These heavy vehicles include a significant number of Type 1 Road Trains and B-Doubles, in addition to Semi-Trailers. Hence, the use of the road for the transport of freight is very important.

Magnitude of the failure

In addition to excess binder on the road surface in the outer wheelpaths, deformation has also begun to occur in the outer wheelpaths.

Over time, both the magnitude and extent of the failures have increased so that much of the 18.5 km long section is affected to some extent. For this reason, rehabilitation work will probably be necessary some time in the future. However, at present roughness and rutting are both only about 60% of the average for the road.

Current risk to road users and risk of deterioration in the future

Overall, at present the road is still very safe, relative to other sections of the Warrego Highway nearby. This is due to both the fact that roughness is still very low, and that the width is much greater than some other road sections. Therefore, rehabilitation could not be justified at present on safety grounds alone.

Investigation Plan and Team

Up until September 2004, some preliminary investigation work has been carried out, to enable the failure cause to be determined, and provide guidance as to how to rectify the failures so that future planned works did not fail due to the same problems.

Visual investigations were first conducted to observe the failure and determine what testing might be necessary to carry out. Some materials sampling and testing has also been completed, to help formulate the failure cause(s) and the best rehabilitation treatment. Further investigation could be expected in the future.

4.7.2 Review Documents and Literature

Plans and Pavement Details

The plans for the job and a typical cross-section of the work were reviewed. This provided information about the work to be carried out and other information. The design life of the project was 20 years, with the design traffic loading being 4.95 x 10^6 ESA's.

The assumed subgrade CBR for design was 6 (after lime stabilisation), and the nominal road crossfall was 3%. The new pavement that was constructed consists of two 3.5m lanes and a 2m shoulder at each side.

Where there was an existing bitumen road (such as along Yaralla Road), the existing pavement was scarified and blended with additional material to bring it to the correct subgrade profile. The subgrade was stabilised to a depth of 250 mm with 3% - 5% of quicklime, depending on location.

The pavement was placed to a total depth of 520 mm. This consisted of 170 mm of a white rock type 4.5 gravel material, 175 mm of a type 3.3 gravel subbase, and 175 mm of a type 3.1gravel base.

The surfacing along the length of the job was predominantly a spray seal, with a 35 mm dense graded asphalt surfacing only being used at some of the 5 intersections. The spray seal included an initial 5 mm prime, followed by an initial seal 10 mm, soon after construction in January 2002. This was followed by a variable rate second 10 mm spray seal in May 2003.

4.7.3 Non-destructive Condition Survey

Visual Examination

Visual examinations of the failures and road section have been conducted from April 2004 onwards, and the most relevant results of these investigations are discussed below.

In late April 2004, an examination was conducted during a rain event. It was observed that there was water ponding near the outer edgelines and wheelpaths. A measurement of the pavement crossfall revealed that the outer pavement had a crossfall of 0.6% to 1.8%, much less than the 3% design value, and it was this, plus the onset of rutting that led to the water ponding.

The change in crossfall near the outer edge of the pavement was probably due to the high shrink-swell black soil swelling up and pushing up the sides of the pavement, so the crossfall was reduced. The stabilisation carried out during construction should have reduced the tendency for volume change, but this does not seem to have occurred.

In July 2004, the road section was examined again, this time during dry weather. By this time, the magnitude and extent of the failures had increased a fair bit, with the failures still being mostly confined to the outer wheelpaths.

The failures varied in both length and extent, with some failures being tens of metres long and other being less than a metre long. The base material near the surface appeared to be getting shoved sideways, and with associated rutting and bleeding.

Figure 4.24 - Yaralla Deviation:

a) and b) Views of the water ponding occurring in the outer wheelpath due to rutting and changes in the crossfall of the pavement

Figure 4.25 – Change in Pavement Crossfall:

a) Location where crossfall had reduced from 3% down to 0.6%

b) Level device is being held at design 3% crossfall, illustrating the reduced pavement crossfall, leading to water ponding

Figure 4.26 – Yaralla Deviation:

a) A typical location with bleeding in the outer wheelpaths

b) A shot illustrating how material in the outer wheelpaths has been shoved sideways, with ridging that hinders the drainage of water off the road surface

PAVDEF Survey

Deflection testing along the Yaralla Deviation using the PAVDEF machine was conducted on two separate occasions. The first run was conducted in December 2001, close to the end of construction in the westbound direction only, from chainage 5.6 – 23.2 km. The second run was conducted in July 2004, in both the eastbound and westbound directions, from chainage 5.0 – 23.6 km.

The data from these tests is provided in the reports *PR 2080A PAVDEF Deflection Survey: Yaralla Deviation* (Queensland Department of Main Roads, 2001a) and *PR 2348A PAVDEF Deflection Survey: Yaralla Deviation* (Queensland Department of Main Roads, 2004a). Graphs of the results are shown in Appendix I.

A brief overview of the results is provided in the tables, with a comparison being made between the 2001 and 2004 test results. The best and worst case test results from the outer and inner wheelpaths is shown.

Table 4.45 – Summary of PAVDEF deflection testing data: Yaralla Deviation – 90% Highest Deflection (mm)

Year / Direction	10% Value	Median (50%)	90% Value
2001 - Westbound	0.29 – 0.35	0.37 – 0.54	0.55 – 0.76
2004 – Westbound	0.37 – 0.55	0.46 – 0.77	0.63 – 1.00
2004 - Eastbound	0.37 – 0.49	0.48 – 0.76	0.61 – 0.97
% Change: 2001 – 2004	28 to 49	27 to 42	13 to 30

Table 4.46 – Summary of PAVDEF deflection testing data: Yaralla Deviation – Mean Curvature (mm)

Year / Direction	10% Value	Median (50%)	90% Value
2001 - Westbound	0.11 – 0.14	0.16 – 0.22	0.24 – 0.34
2004 – Westbound	0.12 – 0.17	0.16 – 0.26	0.22 – 0.36
2004 - Eastbound	0.12 – 0.16	0.16 – 0.26	0.22 – 0.35
% Change: 2001 – 2004	9 to 18	0 to 18	-8 to 4

Table 4.47 – Summary of PAVDEF deflection testing data: Yaralla Deviation – 10% Lowest Subgrade CBR

Year / Direction	10% Value	Median (50%)	90% Value
2001 - Westbound	13 - 16	20 - 23	25 - 25
2004 – Westbound	6 - 9	8 - 12	12 – 16
2004 - Eastbound	6 – 9	8 - 12	13 - 16
% Change: 2001 – 2004	-54 to -44	-60 to -48	-50 to -36

Table 4.48 – Summary of PAVDEF deflection testing data: Yaralla Deviation – 10% Lowest Deflection Ratio

Year / Direction	10% Value	Median (50%)	90% Value
2001 - Westbound	20 - 20	26 - 26	31 – 32
2004 – Westbound	26 - 33	32 - 38	39 – 42
2004 - Eastbound	31 - 32	37 - 38	42 - 42
% Change: 2001 – 2004	43 to 63	33 to 46	31 to 31

These values can be examined to see how well they appear to satisfy the currently available guidelines, as discussed below. It should be noted that the values used for the analysis had already been factored up or down to some extent (e.g. by using 90% highest or 10% lowest values), and this means that the results tend to be on the conservative side.

90% Highest Deflection (mm)

There are no currently available guidelines for this parameter, and it was found that this was strongly related to curvature. Therefore, this parameter will not be considered further, except to point out that the deflection has increased by about 30% from 2001 to 2004, possibly indicating a reduction in the strength of the pavement, especially near the surface.

Mean Curvature (mm)

The Pavement Rehabilitation Manual (Queensland Transport, 1992) states that values greater than 0.4 mm may suggest a lack of stiffness, a thin pavement or a cracked surface, while a value of less than 0.2 mm indicates a stiff pavement.

As can be seen from the above tables, most of the data is below 0.35 mm indicating good stiffness, for both the 2001 and 2004 test results. The change in curvature is small, of the order 10 – 18% for most, indicating a possible reduction in the strength of the pavement, especially near the surface.

10% Lowest Subgrade CBR

The Pavement Rehabilitation Manual (Queensland Transport, 1992) states that for all but bound pavements, the subgrade CBR can be estimated from the deflection results. It is not clear whether a bound pavement also refers to a stabilised subgrade with an unbound pavement such as was used on this job, and so this may have affected the validity of the subgrade estimation.

It can be seen that from 2001 to 2004, the estimated subgrade CBR has reduced by about 40 – 60%. It is possible that this is due to the uncertainty mentioned above, and also could have been due to differing conditions when the testing was carried out. The 2001 test was carried out in December, while the 2004 test was carried out in July, meaning that the soil moisture conditions may have been different, affecting the CBR value.

10% Lowest Deflection Ratio

Baran (1994) suggests the following guidelines for deflection ratio using the PAVDEF Deflection Testing device, expressed as a percentage:

>60	Representative of a bound base
40 – 50	Unbound Granular Base
< 40	Could represent a weakness in the base layer

As can be seen from the above table, the deflection ratio appears to almost satisfactory, with the median value being about 37 – 38 in 2004. It should be noted that there is some conservatism in the results, and that there is no really precise relationship between the value of this parameter and the pavement integrity.

It can be seen that the pavement deflection ration has increased between 2001 and 2004, which is generally an indication of an increase in pavement strength. This is at odds with the deflection and curvature results that increased, indicating a reduction in strength.

Summary

The deflection and curvature results tend to suggest that a reduction in the pavement strength, especially just below the surface, has occurred from 2001 to 2004.

The estimated subgrade CBR values have markedly reduced from 2001 to 2004, although this is probably due to the differing soil moisture conditions at these times, and the uncertainty about the calculation of this value.

The deflection ratio has increased over time from 2001 to 2004, possibly due to continuing strength gains from the lime stabilisation applied to the subgrade. This is at odds with the deflection and curvature results.

Overall, the total pavement strength appears to have increased from 2001 to 2004, but the strength near the road surface, as measured by deflection and curvature, had decreased. This strength loss has been reflected in the failures that have been occurring along the road section, mostly confined to the upper pavement layers.

4.7.4 Destructive Materials Sampling and Testing

Trenching

To help determine the cause of the pavement failures, it was planned to excavate four trenches, to enable material samples to be taken. The location of the trenches was as follows, shown in the table.

Table 4.49 – Location of test trenches on the Yaralla Deviation

Trench Number	Chainage (km)	Direction	Description
1	12.2	Westbound	Unfailed
2	17.1	Westbound	Failed
3	22.2	Westbound	Failed
4	22.5	Eastbound	Failed

During the excavation of the trenches, it was observed that the base gravel appeared to not be overly wet, but did appear to be wetter in the failure area. The type 4.5 white rock material used for the subbase had a consistency similar to plasticine, while the lime stabilised black soil subgrade seemed to be very hard.

Figure 4.27 – Yaralla Deviation trenches:

a) Unfailed location

b) 'Failed' location

Figure 4.28 - Yaralla Deviation Trenches:

a) Trench 4 (‘failed’) with different pavement layers easily visible

b) Trench 1 (‘unfailed’)

Samples of material were taken for testing of the moisture content, and the plasticity index (from the Atterberg Limits). The test results are discussed further below.

Moisture Content Test Results

The moisture content was sampled for each layer at both the edgeline and centre line. The results for the failed trench are contained in the following table.

Table 4.50 – Moisture content results for Yaralla Deviation trench

	Moisture Content (%)	
Layer	**Near edgeline**	**Near centre line**
Type 3.1 / 3.3 Base	5 – 6	3.5 – 5.5
Type 4.5 White Rock Subbase	15 – 23	12 – 17
Stabilised Black Soil Subgrade	15 - 24	5 – 21

As can be seen from the above results, all pavement layers are wetter near the edge line, possibly indicating that moisture is entering the pavement from the sides.

Plasticity Index Test Results

The material used for testing and calculation of the plasticity index of the base was sampled from near the edgeline, with samples for the top 50 mm and the bottom 125 mm being separately tested. The results are shown in the following table, along with the required specification values.

As can be seen from the results, the plasticity index in the top 50 mm of the base is higher than that for the other 125 mm below in all trenches. It is also interesting to compare the % difference in the top part, compared to the rest of the layer.

In trench 1, where there was little or no failure occurring, the increase in the plasticity index near to the surface is only about 10%. In the other 'failed' trenches, the i ncrease in the plasticity index is much higher, ranging from 20 – 40%. This result appears to indicate that the failure is related to the increase in the plasticity index.

From standard specification MRS 11.05 – Unbound Pavements, the maximum plasticity index for a type 3.1 material is recommended as 6. None of the samples satisfy this criterion.

Table 4.51 – Plasticity index test results for base material: Yaralla Deviation trench

	Plasticity Index in type 3.1 base material		
Trench Number	**Top 50mm**	**Bottom 125mm**	**% Difference**
1	9.0	8.2	10
2	9.2	7.6	21
3	10.4	7.4	41
4	8.6	6.6	30

CBR Test Results

CBR tests were conducted on material samples from the base, subbase, white rock gravel, and lime stabilised subgrade. The test results are summarised in the table.

Table 4.52 – CBR results for Yaralla Deviation trench

Layer	CBR Values	Specification MRS 11.05 - CBR (Soaked)
Type 3.1 Base	42, 9, 23, 40	80
Type 3.3 Subbase	11, 14, 34, 7	45
Type 4.5 White Rock	14	15
Black Soil Subgrade	All > 60	Design subgrade of 6

As can be seen, the base and subbase CBR values are much less than the specification values, indicating possibly some weakness in these structures. The CBR for the white rock is almost okay, while that for the lime stabilised subgrade is much higher than required.

4.7.5 Determine Probable Cause(s) of Failure

The above section of the report details all information regarding the project, the failures occurring and the investigation and testing carried out.

Next all possible failure causes were considered, along with any information that either supports or refutes each hypotheses. Since the failure occurred first as bleeding and flushing and rutting is starting to occur, the possible cause(s) of each of these failure types will be examined.

Table 4.53 – Possible failure causes for bleeding and flushing, with associated information

Possible Failure Cause	Information
Too much binder sprayed	No evidence
Insufficient surface aggregate	Not observed during visual investigation
Non-uniformity/patching of original surfacing	Unlikely, since new construction
Embedment of surface aggregate, due to weak base	Plasticity of top base has increased, particularly in failed sections. CBR values are also low.
Lack of proper rolling	No evidence
Failure to protect from traffic for long enough	No evidence
Loss of surface aggregate due to stripping or ravelling	Not observed during visual investigation
Breakdown of surface aggregate	No evidence, although breakdown/degradation of base material may be occurring

Possible Failure Cause	Information
Poor spreading of aggregate	Not observed during visual investigation
Over-filled voids in asphalt	Not applicable
Lack of size of aggregate leading to covering by binder	No evidence

Table 4.54 – Possible failure causes for rutting, with associated information

Possible Failure Cause	Information
Inadequate pavement thickness	No evidence, since pavement is quite thick
Weak subgrade	CBR values for subgrade indicate strength is higher than design
Weak base	Plasticity and CBR tests indicate that degradation of base material may be occurring
Surfacing lack of strength / stability	No evidence
Inadequate Compaction	No evidence
Poor Material Quality	Base material seems to be undergoing degradation, based on testing
Excessive Moisture	Testing indicates moisture is higher near outer edgeline where failing, compared to near centreline
Very high traffic loading (> 10^7 ESAs)	Design traffic is not excessive
Inappropriate mix design (asphalt)	Not applicable
Interaction between layers	No evidence

Probable Cause(s) of Failure

From the above tables, it can be seen that the probable failure cause has been identified as a weakness in the base material.

It is likely that the bleeding of the surface is occurring due to embedment of the surface aggregate into the base material. The weakness of the base is shown by the low CBR values from the materials testing, and the increase of the plasticity index near the top of the base, and by the pavement deformation and rutting that is starting to occur in the outer wheelpaths.

The test results for the subgrade and type 4.5 white rock indicate that these layers are probably satisfactory, with little weakness contributing to the failure. However, it seems as if the black soil subgrade, while stabilised, still changes volume allowing changes in the pavement cross fall and water ponding on the road.

The weakness of the base material may be due to an increase in moisture content near the outer wheelpaths (where the failures are occurring), compared to near the road centreline. It does seem as if moisture is entering the pavement, and causing or contributing to the weakness of the base material.

4.7.6 Determine Best Rehabilitation Treatment

Possible Rehabilitation Treatments

Possible rehabilitation treatments would be similar to those shown previously for the Bowenville-Dalby section, consisting of the following possible treatments:

- Full width asphalt overlay
- Full width stabilisation
- Full width granular overlay

- Stabilise outer wheelpaths
- Asphalt in outer wheelpaths
- Granular material in outer wheelpaths

The relative costs of these treatments would also be similar to those shown for Bowenville-Dalby. In addition, or as part of the treatment, it may also be necessary to increase the pavement cross fall to reduce the effect of volume change of the black soil subgrade.

The selection of which treatment should be used in the future would be dependent on the rate at which the failure increases in both extent and magnitude. Further materials testing would be required before a firm decision could be made.

Chapter 5

Conclusions

5.1 Summary of Investigation Method

The investigation methodology developed in this project contained the following steps:

- Plan the Investigation
- Review Documents and Literature
- Interview Personnel
- Non-destructive Condition Survey

- Destructive Materials Sampling and Testing
- Determine Probable Cause(s) of Failure
- Determine Best Rehabilitation Treatment
- Report on Outcomes

When planning the investigation, a general review of the problem should first be conducted, along with the possible scope of investigation and rehabilitation work that may need to be carried out. An investigation plan should be drafted, addressing goals, budgeting constraints, operations planning and the investigative synthesis. The investigation team should be decided upon.

Reviewing documents and literature may involve the inspection of plans, pavement history, drainage design, pavement materials information and specifications, previous material test results, construction records, testing methods and frequencies, and other relevant information, such as publications.

Personnel that may need to be interviewed may include designers, construction or maintenance personnel, and any other personnel that may have useful information.

The non-destructive condition survey may include a visual examination of the pavement failure, the effectiveness of drainage structures and other details. Deflection testing of the pavement may be carried out, and while the interpretation of the information gained is somewhat difficult, useful information can be gathered. Other forms of non-destructive testing may also be carried out as required.

Destructive materials sampling and testing may be carried out on the pavement failure if it is deemed to be necessary in providing more information about the cause(s) of the pavement failure and possible rehabilitation treatments. Trenching or coring may be used to provide material samples for laboratory testing, and also allows a visual examination of the pavement structure. Destructive testing may include tests on the soils and aggregate, geotechnical tests, tests on asphalt and tests on bituminous materials.

When determining the probable failure cause(s), it is normally impossible to say with complete certainty what the cause(s) of the pavement failure was, and there are often multiple factors that contributed to the failure. The process involves investigative synthesis, determining which information supports or refutes each of the possible failure hypotheses, and determining the probable cause(s) of the failure based on this.

When selecting the best rehabilitation treatment, it is necessary to subject possible alternatives to a detailed examination of both economic and other factors. Rehabilitation options may be split up into those for granular pavements, asphalt pavements, and rehabilitation of the moisture control or drainage system. Treatments may include surface treatments, overlays, in-situ stabilisation, or other miscellaneous rehabilitation treatments.

A report on the outcome of the pavement failure investigation should be produced, as this enables others to learn from the failure, and should help reduce the chances of a similar failure in the future. Information that should be included is a general review of the project and location, failure details, a description of any testing carried out, what the probable cause(s) of failure were, how it could be prevented in the future, and possible rehabilitation options.

5.2 Project Conclusions

Forensic engineering is the application of the engineering sciences to the investigation of failures or other performance problems, with a focus on uncovering the causes of failures so that improved facilities can be engineered.

The investigation of road pavement failures can be done in a systematic manner using the principles of forensic engineering. The methodology developed in this project has been based on similar work previously conducted in various locations, mainly in Australia and the United States. The focus is on creating a systematic, and yet simple and easy to understand guide that is flexible enough for use in a variety of situations.

The investigation methodology developed was trialled in several pavement failures in Southern District of the Queensland Department of Main Roads, to evaluate the effectiveness of the method for real use.

It was found that the method was good as a general guide, particularly for people inexperienced in the area of pavements and road engineering. However, the experience of the investigator is also an important factor in correctly diagnosing the pavement failure cause and determining the best rehabilitation treatment.

This is because it is very difficult to list all possible types of failure causes and understand how they interact. Knowledge of how this occurs for a particular pavement type and environmental conditions can often only be determined through years of experience in this area.

The study showed that for most pavement failures it is necessary to carry out some form of materials sampling and testing, if conclusive evidence regarding the failure cause is to be found. Otherwise, interpretation of the limited data available can be difficult, or even impossible.

This is due to the wide variety of causes and types of pavement failure, and the normal lack of visual or documented information that may indicate which of these causes is the failure cause. While it is often possible to evaluate the most likely failure cause without materials sampling and testing, this hypotheses is essentially unproven until testing is carried out, meaning that it could be wrong, leading to incorrect decisions being made about rehabilitation of the pavement.

The pavement failure investigation methodology developed in this project can serve as a useful guide for the investigation of pavement failures. The method, combined with the experience of the investigator and adequate materials investigation, will help to ensure that the cause of a pavement failure can be reliably determined.

5.3 Achievement of Project Objectives

The objectives of this project were discussed in Chapter 1 – Introduction, and are also listed in Appendix A. The following paragraphs discuss the achievement of the objectives.

The first objective was to research background information on forensic engineering and pavement engineering, and in particular the history and methods of forensic engineering, application of forensic engineering in the road engineering area, and pavement assessment, testing and failure modes.

This objective was achieved by a comprehensive review of pavement assessment techniques, testing and failure modes, with information coming from a variety of sources, both in Australia and overseas. This information formed an integral part of the investigation method that was later developed.

This phase of the investigation showed that the principles of forensic engineering do not appear to be widely understood or used in the engineering profession. However, this situation is changing, due to an increased understanding of the importance of forensic engineering for the rehabilitation and maintenance of all engineering structures, including road pavements.

The second objective was to develop a systematic method for the forensic investigation of pavement failure that is applicable to the Toowoomba District. The method developed was based on previous attempts by others, but with a focus on developing a method that is simple, easy to use, and yet flexible enough to be used in a variety of situations.

The method was confined to the examination of pavement failures in unbound granular and asphalt pavements, due to the prevalence of these pavement types on Australian roads and in the Southern District of the Queensland Department of Main Roads. However, the principles involved in the method could easily be transferred to other pavement types, with only the failure types and causes needing to be modified.

The strong point of the method is its simplicity and ease of use, while covering all necessary steps required for a pavement failure investigation. One weak point of the method is the lack of technical detail contained in it. However, the reader is referred to other publications where appropriate, and considering the current state of change in materials characterisation and testing methods, this is not considered to be a major limitation.

The third objective was to select, in conjunction with Main Roads staff, a number of failing pavements within the Toowoomba District. This objective was achieved by consultation with relevant personnel, and the sites selected were all fairly young, and could therefore be considered as premature pavement failures.

The fourth objective was to apply the method to the selected sites. There were some difficulties in this area, due to limitations in the materials sampling and testing carried out. However, this limitation is probably an indication of how the method would really be applied in practice, with more minor failures not being tested to any great extent due to budget constraints.

The application of the method to the selected sites in some cases helped to change the thinking on what was happening in the failures. For example, the use of trenching for the collection of material samples on the Bowenville-Dalby job helped to reveal that the problem was confined to the top 50 - 100mm of the base material. Without trenching and materials sampling, this may not have been discovered until the failure had progressed much further.

The fifth objective was to analyse the information obtained from each site and determine if adjustment to the original method is needed. Since the adjustment of the method had been a continuous process throughout the work, after applying the method to the selected sites, no major changes were needed.

The sixth objective was to determine if the forensic method developed is appropriate for use in the Toowoomba District, from both a technical viewpoint and an economic viewpoint, and its applicability to the future. The method developed would be appropriate for use in both Toowoomba District, and other locations.

Technically, the detail provided would need to be supplemented by other publications, but the method would be a useful guide in the future. Economically the method allows the flexibility of not including particular steps in the investigation if the failure is fairly minor. In addition, it will help to ensure that unnecessary testing is not carried out, and that members of the investigation team only meet at required times, therefore reducing personnel costs.

The seventh objective was to report findings through oral presentation at the Project Conference, and in the required written format. The conference presentation appeared well received with a good-sized audience and a probing question time. This document, the dissertation, represents the written format of the project.

5.4 Further Work

The investigation methodology developed in this project was quite general, with regard to technical details. Further work on the topic that could be conducted in the future might include the following:

- Detailed examination of pavement failures

- Detailed examination of the relationship between deflection testing and actual pavement performance

- Examination of the advantages of performance based testing over traditional empirical testing methods

The pavement failure causes outlined in this project were quite general, and could be examined in more detail, with a view to increased understanding of the failure causes and contributing factors. This could be simulated at a small scale in a testing lab, although interpreting the data in terms of a full size road pavement may be difficult.

Another possibility is to utilise the method developed in this project, and continue to use this systematic forensic methodology to examine pavements where failure is occurring, thus reinforcing the importance of forensic engineering for the investigation of pavement failures.

Deflection testing is quite open to interpretation and a project in the future could provide further detail about the relationship between deflection results and actual pavement performance. This could be done either in the lab or by examining actual pavement sites, possibly using testing previously carried out by the Queensland Department of Main Roads.

Another possibility for further work is a study of the advantages of performance based testing over traditional empirical testing methods. This could be done by testing small-scale road samples using both empirical and performance based methods, and comparing the results when the samples are loaded until significant failure occurs.

For example, a number of pavement samples could be constructed and tested using CBR, Atterberg Limits and Repeat Load Triaxial tests. These samples could then be loaded, allowing an examination of how rutting or pavement deformation varies with these test material test results.

List of References

APAI, 2003, ***Appendix A -Pavement Failure Identification, Identifying and correcting pavement failures***, 2003, Asphalt Paving Association of IOWA, viewed 29 January 2004, <http://www.apai.net/apai2003/designguide/Appendix_A-B.pdf>

Standards Australia, 2002, ***AS1289.0 – Testing of Soils for Engineering Purposes – General Requirements and List of Methods***, Australia

Austroads, 2000a, ***Guide to Selection of Road Surfacings***, Austroads Incorporated, Sydney

Austroads, 2002a, ***Austroads Pavement Strategy 2001-2004***, Austroads Incorporated, Sydney

Austroads, 2003a, ***Pavement Strength in Network Analysis of Sealed Granular Roads: Basis for Austroads Guidelines***, Austroads Incorporated, Sydney

Austroads, 2003b, ***Development of Performance-based Specifications for Unbound Granular Materials – Part A: Issues and Recommendations***, Austroads Incorporated, Sydney

Austroads, 2003c, ***Development of Performance-based Specifications for Unbound Granular Materials – Part B: Use Of RLT Test To Predict Performance***, Austroads Incorporated, Sydney

Austroads, 2004, ***Salinity and Rising Water Tables — Risks for Road Assets***, Austroads Incorporated, Sydney

AusStab, 2000, ***Site Investigations for the Rehabilitation of low-trafficked roads using In-situ Recycling***, Australian Stabilisation Industry Association, viewed 29 January 2004, <http://www.auststab.com.au/nat07.pdf>

Baran, E, 1994, ***Developments in Pavement Testing and Interpretation*** [Paper presented at Queensland Transport's Workshop on Low Volume Roads, Longreach, March, 1994]

Campbell-Allen, D, 1987, '**Forensic Engineering – A Need in Australia?**' [Paper C1707 submitted to The Institution of Engineers, Australia], Civil Engineering Transactions, 1988, pp.32-35

Carper KL, 1989, ***Forensic Engineering***, Elsevier Science Publishing, USA

Chen, D-H, Scullion, T, Bilyeu, J, Yuan, D & Nazarian, S, 2002, '**Forensic Study of Warranty Project on US 82**', Journal of Performance of Constructed Facilities, Feb 2002, Vol. 16, No. 1

Chen, D-H, Bilyeu, J, Scullion, T, Lin, D-F, & Zhou, F, 2003, '**Forensic Evaluation of Premature Failures of Texas Specific Pavement Study – 1 Sections**', Journal of Performance of Constructed Facilities, May 2003, Vol. 17, No. 2, pp. 267-74,

Chen, D-H, Scullion, T, Bilyeu, J, Fults, K & Murphy, M, ***Pavement Forensics, Investigating Failures***, 'Better Roads', January 2004, viewed 29 January 2004, <http://www.betterroads.com/articles/jan04d.htm>

Crampton, D, Zhang, Z, Fowler, DW & Hudson, WR, 2001, ***Development of a formal Forensic Investigation Procedure for Pavements*** [Report no. FHWA/TX-03/1731-3F, Center for Transportation Research, University of Texas], viewed 23 June 2004, < http://www.utexas.edu/research/ctr/pdf_reports/1731_3F.pdf

Demos G, 2003, ***Appendix A - Evaluating Premature Distress of Hot Mix Asphalt and Portland Cement Concrete Pavements***, Colorado Department of Transportation, viewed 29 January 2004, <http://www.dot.state.co.us/DesignSupport/PavementDesignManual/New%20Pavement%20Design%20Manual%208-13-03/Appendix%20A.pdf>

Dowling L, 1998, ***Pavement Management***, Deakin University and Australasian Pavement Studies and Research Centre (APSARC), Australia

Gabr, MA, Hopkins, K, Coonse, J & Hearne T, 2000, '**DCP Criteria for Performance Evaluation of Pavement Layers**', Journal of Performance of Constructed Facilities, Nov 2000, Vol. 14, No. 4

HAPI, ***Pavement Distress,*** Hawaii Asphalt Paving Industry, viewed 20 September 2004, <http://www.hawaiiasphalt.com/HAPI/modules/03_general_guidance/03_pavement_distress.htm>

Investigation of Failures on Job 67/18B/302.1, 2004, Infrastructure Delivery, Southern District, Queensland Department of Main Roads, Toowoomba

Main Roads (WA), 2004, ***Test Methods***, Main Roads (Western Australia), viewed 23 August 2004, http://www.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={1532D87F-C1AC-4386-9968-5E5F4FD002E5}

Mooney, MA, Miller, GA, Teh, SY & Bong, W, 2000, '**Importance of Invasive Measures in assessment of existing pavements**', Journal of Performance of Constructed Facilities, Nov 2000, Vol. 14, No. 4

Muller, W, 2001, ***Ground Penetrating Radar (GPR) - Its applications within Main Roads and results of recent Concrete and Timber Field Trials*** [presented at Road System and Engineering Technology Forum 2001], Road System and Engineering Group, Queensland Department of Main Roads

Muller, W, Jones, J & Kendall, M, 2001, ***Trial Ground Penetrating Radar (GPR) Pavement Investigations in Border District*** [presented at Road System and Engineering Technology Forum 2001], Road System and Engineering Group, Queensland Department of Main Roads

Neale, BS (ed.) 1998, ***Forensic engineering: a professional approach to investigation*** (proceedings of the international conference organised by the Institution of Civil Engineers and held in London, UK, on 28-29 September 1998), Thomas Telford, London

Noon, RK, 2001, ***Forensic Engineering Investigation,*** CRC Press LLC, Florida

Park, S-W, Lytton, RL & Button, JW, 1999, '**Forensic Investigation of Pavement Distortions using Soil Suction'**, Journal of Transportation Engineering, Jan-Feb 1999, Vol. 125, No.1

Proposed Bulk Maintenance Treatment – Drayton Connection Road (2003), Infrastructure Delivery, Southern District, Queensland Department of Main Roads, Toowoomba

Queensland Department of Main Roads, 1989a, ***Technical Note 4 – Pavement Evaluation Procedures***, Transport Technology, Queensland

Queensland Department of Main Roads, 1989b, ***Technical Note 6 – Design of Overlays***, Transport Technology, Queensland

Queensland Department of Main Roads, 1993, ***Technical Note 7 – Controlling Moisture in Pavements***, Transport Technology, Queensland

Queensland Department of Main Roads, 1999, ***Standard Specifications Roads***, viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/Prod/Content.nsf/DOCINDEX/Road+Related?OpenDocument>

Queensland Department of Main Roads, 2001a, ***PR 2080A PAVDEF Deflection Survey: Yaralla Deviation,*** Pavement Testing, Queensland

Queensland Department of Main Roads, 2002a, ***Materials Testing Manual: List of Test Methods***, Queensland, viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/prod/CONTENT.NSF/0/8105fadfcb3f2d9c4a256d27001ed64f/$FILE/MTM%20Contents%2002.pdf>

Queensland Department of Main Roads, 2002b, ***Road Drainage Design Manual,*** Queensland, viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/Prod/Content.nsf/b495dab138a6b17a4a256a42001c8f4f/c1decd6de87275fa4a256df3000c6214!OpenDocument>

Queensland Department of Main Roads, 2002c, ***Asset Maintenance Guidelines***, Queensland, viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/Prod/Content.nsf/DOCINDEX/Asset+Maintenance+Guidelines?opendocument>

Queensland Department of Main Roads, 2003, ***PR 2254D PAVDEF Deflection Survey: Gatton Bypass Duplication,*** Pavement Testing, Queensland

Queensland Department of Main Roads, 2004a, ***PR 2348A PAVDEF Deflection Survey: Yaralla Deviation,*** Pavement Testing, Queensland

Queensland Department of Main Roads, 2004b, ***PR 2331A PAVDEF Deflection Survey: Warrego Highway through Toowoomba,*** Pavement Testing, Queensland

Queensland Department of Main Roads, 2004c, ***Gatton Bypass Duplication - Analysis of FWD Deflection Results***, Pavements, Materials and Geotechnical Division, Queensland

Queensland Transport, June 1992, ***Pavement Rehabilitation Manual***, Pavement and Assets Strategy Branch, Queensland

Ramanujam, J, 2001, ***Risk Profiles in Pavement Rehabilitation*** [presented at Road System and Engineering Technology Forum 2001], Road System and Engineering Group, Queensland Department of Main Roads

Reeves, I, 2001, ***Risk Management – Materials*** [presented at Road System and Engineering Technology Forum 2001], Road System and Engineering Group, Queensland Department of Main Roads

Rens KL (ed.) 1997, ***Forensic engineering: Proceedings of the First Congress: October 5-8, 1997***, Minneapolis, Minnesota, American Society of Civil Engineers, Virginia USA

Rens KL (ed.) 2000, ***Forensic engineering: Proceedings of the second congress, May 21-23, 2000***, San Juan, Puerto Rico, American Society of Civil Engineers, Virginia USA

Roads and Traffic Authority of New South Wales, 1998, ***Materials QA Specifications***, viewed 23 August 2004, <http://www.rta.nsw.gov.au/doingbusinesswithus/specifications/materials.html>

Roads and Traffic Authority of New South Wales, 2003, ***Materials Test Methods – Volumes 1, 2 & 3***, viewed 23 August 2004, <http://www.rta.nsw.gov.au/doingbusinesswithus/specifications/volume1materialstestmethods.html>

Scala, AJ, 1979, ***An Analysis of the Deflection Bowls in Pavements Measured by the Benkelman Beam Test***, ARRB Internal Report AIR 032-2.

Victorine, T, Zhang, Z, Fowler, DW & Hudson, WR, 1997, ***Basic concepts, current practices, and available resources for forensic investigations on pavements*** [Report no. FHWA/TX-98/1731-1, Center for Transportation Research, University of Texas], viewed 6 March 2004, <http://www.utexas.edu/research/ctr/pdf_reports/1731_1.pdf>

Vic Roads, 2004b, ***Testing Methods Index***, viewed 23 August 2004, <http://www.vicroads.vic.gov.au/vrne/VRbsCat.nsf/CatalogByCategory/30C6BE7248EDAC49CA256CED007FC081!OpenDocument&CartID=15890-033050>

Woods, WR & Adcox Jr, AW, 2002, '**A General Characterization of Pavement System Failures, with Emphasis on a Method for Selecting a Repair Process'**, Journal of Construction Education, Vol. 7, No. 1, pp. 58-62, viewed 29 January 2004, <http://ascjournals.org/jce/journal/2002/no1/Spring%202002,%20Vol.%207,%20No.%201,%20pp.%2058-62.pdf>

WAPA, ***Surface Distresses,*** Washington Asphalt Pavement Association, viewed 20 September 2004, <http://www.asphaltwa.com/wapa_web/modules/11_guidance/11_surface_distress.htm>

WSDOT, ***Pavement Distress***, Washington State Department of Transportation, viewed 28 June 2004, <http://hotmix.ce.washington.edu/wsdot_web/Modules/09_pavement_evaluation/09-7_body.htm>

Bibliography

AAPA and Austroads, 2004, ***Pavement Work Tips***, Australian Asphalt Pavement Association, viewed 23 August 2004, <http://www.aapa.asn.au/docs/publications_worktips.html>

Austroads, 2000b, ***Austroads Specification Framework for Polymer Modified Binders***, Austroads Incorporated, Sydney

Austroads, 2000c, ***Austroads Provisional Specification for Multigrade Binders***, Austroads Incorporated, Sydney

Austroads, 2001b, ***Austroads Provisional Sprayed Seal Design Method Revision 2000***, Austroads Incorporated, Sydney

Austroads, 2002b, ***Integrated Asset Management Guidelines for Road Networks***, Austroads Incorporated, Austroads Incorporated, Sydney

Austroads, 2002c, ***Austroads Framework for Specifying Asphalt***, Austroads Incorporated, Sydney

Austroads, 2002d, ***Guide to Road Profile Unevenness and Bridge Damage***, Austroads Incorporated, Sydney

Austroads, 2003d, ***Development of Performance Contracts and Specifications - Summary Report***, Austroads Incorporated, Sydney

Austroads, 2003e, ***Capturing the Benefits of Performance Based Standards***, Austroads Incorporated, Sydney

Austroads, 2003f, ***Remaining Life of Road Infrastructure Assets: An Overview***, Austroads Incorporated, Sydney

Bell A, 2003, ***Pavements, Materials & Geotechnical Aspects in High Risk Environments*** [presented at 2003 Workshop on Soils in Rural Queensland], viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/Prod/Content.nsf/DOCINDEX/Conferences++Forums+and+Workshops?OpenDocument>

Bruscella, B, Rouillard, V & Sek, M, 1999, '**Analysis of Road Surface Profiles**', Journal of Transportation Engineering, Jan-Feb 1999, Vol.125, No. 1

Caltrans, 2001, ***Flexible Pavement Rehabilitation Manual***, California Department of Transportation, viewed 7 June 2004, <http://www.dot.ca.gov/hq/esc/Translab/pubs/RehabManual-June2001.pdf>

Dastich, I & Dawson, D, 2003, ***Techniques for assessing the non-linear response of unbound granular layered structures***, University of Nottingham, viewed 28 February 2004, <http://www.nottingham.ac.uk/~evzard/ingopap.pdf>

Dawson, A, 2001, ***Granular Pavement Layer Materials ... Where Are We?,*** University of Nottingham, viewed 28 February 2004, <http://www.nottingham.ac.uk/~evzard/arrb2001wkshp.pdf >

Edwards, D, Hogan, S and Kapitzke, J, 2000, ***Technical Note WQ36 – Sealing and Surfacing Practice in Western Queensland (1999 Practice)***, Road System and Engineering Group, Queensland Department of Main Roads

Friedrichs, DA, 2002, ***Pavement design and programming options: European design to maintain only***, Wisconsin Department of Transportation, viewed 29 January 2004, <http://www.dot.wisconsin.gov/library/research/docs/finalreports/tau-finalreports/maintain.pdf>

Kapitzke J & Patane J, 2000, ***Technical Note WQ34 – Traffic of Western Queensland***, Road System and Engineering Group, Queensland Department of Main Roads

Queensland Department of Main Roads, 2000, ***Technical Note 28 – Material Transfer Vehicles in Asphalt Paving towards smoother and longer lasting pavements***, Transport Technology, Queensland

Queensland Department of Main Roads, 2001b, ***Technical Note 8 – Paving Geotextiles in Asphalt and Sprayed Seal Surfacings***, Queensland

Summers, CJ, 2003, ***Road pavement and road surface failure***, viewed 28 January 2004, <http://www.highwaysmaintenance.com/failtxt.htm>

Wijeyakulasuriya, V & Ramanujam, J, 2003, ***Characteristics of Unbound Granular Materials*** [presented at Road System and Engineering Technology Forum 2003], Road System and Engineering Group, Queensland Department of Main Roads

Wilson, PR, 2003, ***Understanding Soil Profiles - Description & Classification*** [presented at 2003 Workshop on Soils in Rural Queensland], viewed 23 August 2004, <http://www.mainroads.qld.gov.au/MRWEB/prod/CONTENT.NSF/0/8ef26f45ba5c70c34a256d74000d6503/$FILE/03_P_Wilson.pdf>

Vic Roads, 2004a, ***GeoPave Technical Notes***, viewed 23 August 2004, <http://www.vicroads.vic.gov.au/vrne/vrbscat.nsf/CatalogByTitle?SearchView&Query=geopave&SearchMax=0&SearchOrder=4&start=0&CartID=>

Appendix A

Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

<u>ENG 4111/2 Research Project</u>

PROJECT SPECIFICATION

FOR:	Richard Smith
TOPIC:	**Forensic Investigation of Pavement Failures**
SUPERVISOR:	Associate Professor Ron Ayers
SPONSORSHIP:	Department of Main Roads, Queensland

PROJECT AIM:

The project seeks to develop and apply a forensic method for the investigation of pavement failures in Toowoomba District, Department of Main Roads, Queensland.

PROGRAM:

- Research background information on Forensic Engineering and Pavement Engineering, and in particular:
  - history and methods of forensic engineering;
  - application of forensic engineering in the road engineering area; and
  - pavement assessment, testing and failure modes.

- Develop a systematic method for the forensic investigation of pavement failure, which is applicable to the Toowoomba District.

- Select, in conjunction with Main Roads staff, a number of failing pavements within the Toowoomba District. Initial investigation will be on about 8 sites.

- Apply the method to the selected sites.

- Analyse the information obtained from each site and determine if adjustment to the original method is needed.

- Determine if the forensic method developed is appropriate for use in the Toowoomba District, from both a technical viewpoint and an economic viewpoint, and its applicability to the future.

- Report findings through oral presentation at the Project Conference, and in the required written format.

Appendix B

Assessment of Effects of Project

Sustainability Effects

This is best assessed using the document: Towards Sustainable Engineering Practice: Engineering Frameworks for Sustainability (Institution of Engineers, Australia, Canberra, 1997).

Development today shall not undermine the development and environmental needs of future generations. What impact might the project or its results have on the usage of finite resources and waste production?

Since the project essentially will utilise existing testing techniques the impact should be minimal, or at least no worse than that currently associated with road construction and maintenance. In addition, if premature pavement failures can be prevented through increased understanding of pavements, less resources will be 'wasted' fixing up failures, and resources used in pavements should be able to be used longer without failure occurring.

Environmental protection shall constitute an integral part of the development process. Identify the environmental protection dimension of the project that shall need to be accommodated in your designs.

There is no environmental protection dimension that must be a part of the development process. This is due to the similarity to existing testing methods. In addition, if any environmental degradation does occur, this should be negligible when considered relative to that which occurs during road construction and maintenance, and usage by vehicles.

Engineering people should take into consideration the global environmental impacts of local actions and policies. Assess the future (global) impact of the project or what might flow from it in future years (i.e. not just immediate impacts).

This method is already being used to a certain extent in some other parts of the world (notably Texas, USA). Since the method being developed will aim to be specific to Southern district, it would need to be adapted for use in other places.

However, an increase in the understanding of the behaviour of pavements will help to ensure that less resources are used fixing up failures, and that resources used for pavements will hopefully last longer without failing. This should help to ensure that the finite supply of resources available for road engineering is used as fully as possible.

The precautionary approach should be taken – scientific uncertainty should not be used to postpone measures to prevent environmental degradation. General rule of caution is: If in doubt, don't. However, a proactive approach may be expected if current practice is unsuitable. Does the need for action despite incomplete or uncertain knowledge apply to the project?

As mentioned above, since forms of the method are already in use, there should not be any great problem with use of the method in Southern district.

Environmental issues should be handled with the participation of all concerned citizens. Assess who might be impacted (or who might perceive an impact) and set out a plan for their involvement.

The groups most likely to be affected by this method are road users, road authorities (specifically Depart of Main Roads, Queensland) and the general community. Road users are unlikely to be greatly concerned with the method, but if use is successful, they will probably appreciate the better and longer-lasting roads that may result.

Road Authorities (Main Roads) will give feedback regarding the project while it is carried out, and this should ensure the method is acceptable to them. The project may result in less money being spent on maintenance, which could possibly be spent on other areas, such as construction or upgrading of existing roads.

The general community may benefit from the project. This is because if longer-lasting better roads result from increased understanding of pavements, this money could conceivably be spent on other areas such as health or education.

The community has a right of access to, and an understanding of, environmental information. Structure appropriate parts of project appreciation and dissertation so that these aspects of the work are readily accessible and readily understandable.

As mentioned above, any environmental impact should be minimal, and as such this is not very applicable to the project.

The polluter should bear the cost of pollution and so environmental costs should be internalised by adding them to the cost of production. Identify these (potential) costs, even if they will not form part of project cost this year.

Pollution caused by application of the method should be minimal when compared with that caused by road users. Since almost none of the Australian community are self sufficient, and are to a certain extent dependent on roads and the freight transported along them, the cost of pollution must be borne by the whole community, both now and increasingly in the future.

The eradication of poverty, the reduction in differences in living standards and the full participation of women, youth and indigenous people are essential to achieve sustainability. Identify potential impact of the project on these groups: if answer is nil, this must be justified, and consider saving of labour.

The impact of the project on women, youth and indigenous people will be minimal. This is because the method is essentially a 'unification' of existing testing methods already in use. Better and longer-lasting roads will help to ensure that differences in living standards between urban and rural people in the Australian community are kept as low as possible.

People in developed countries bear a special responsibility to assist in the achievement of sustainability. Examine the scenario in which outcomes of project work are utilised in both developed and undeveloped countries of the world. Are sustainability outcomes the same, or are there differences?

Generally, the use of the method will depend on what stage the country's road network is currently at. In a developing country, the focus will be on building the roads and developing the network.

In contrast, in developed countries such as Australia, the focus is not just building the road, but rather on building and maintaining the road network at the lowest possible cost, while still providing an acceptable standard of safety and service. The method however, should be equally applicable to both developing and developed countries, although it could be used in different ways and for different reasons.

Warfare is inherently destructive of sustainability, and in contrast, peace, development and environmental protection are interdependent and indivisible. - How might the project, and/or its outcomes, contribute to international understanding?

The effect of the project on warfare will be minimal, as will be its effect on international understanding.

Ethical Effects

This is best assessed by considering the nine tenets of the IEAust (Institution of Engineers Australia) Code of Ethics. This was downloaded from the following website: http://www.ieaust.org.au/directory/res/downloads/Code_of_Ethics_2000.pdf

Members shall at all times place their responsibility for the welfare, health and safety of the community before their responsibility to sectional or private interests, or to others members.

Better roads will be of benefit to the community, in terms of welfare, health and safety.

Members shall act in order to merit the trust of the community and membership in the honour, integrity and dignity of the members and the profession.

The project will not be kept secret, and could be used by anyone.

Members shall offer services, or advise on or undertake engineering assignments, only in areas of their competence and shall practise in a careful and diligent manner.

The method developed will have feedback from Main Roads employees with experience in the area of road engineering.

Members shall act with fairness, honesty and in good faith towards all in the community, including clients, employers and colleagues.

The project will not be kept secret, and could be used by anyone.

Members shall apply their skills and knowledge in the interest of their employer or client for whom they shall act as faithful agents or advisers, without compromising the welfare, health and safety of the community.

While the project and method are being developed partly in conjunction with Main Roads, this is a public-sector organisation and as such, essentially is owned by the 'taxpayer'. Standards of accountability will ensure any work done will benefit the community.

Members shall take all reasonable steps to inform themselves, their clients and employers and the community of the social and environmental consequences of the actions ad projects in which they are involved.

The Project is essentially a unification of existing methods and as such, social and environmental consequences should be minimal, especially when being compared with those caused by road users.

Members shall express opinions, make statements or give evidence with fairness and honesty and on the basis of adequate knowledge.

The method developed will have feedback from Main Roads employees with experience in the area of road engineering.

Members shall continue to develop relevant knowledge, skill and expertise throughout their careers and shall actively assist and encourage those under their direction to do likewise.

Not really applicable, although increased understanding of pavements will benefit all engineers who work in the road engineering area.

Members shall not assist, induce or be involved in a breach of these Tenets and shall support those who seek to uphold them.

Not applicable.

Appendix C

Project Methodology and Timelines

Project Methodology

The specific objectives as listed earlier will now be considered in terms of how each task will be accomplished, and why the decision was made to carry out the work this way (if relevant).

1) Research background information on Forensic Engineering and Pavement Engineering, and in particular:

History and methods of forensic engineering

Application of forensic engineering in the road engineering area

Pavement assessment, testing and failure modes

This will be achieved by conducting a literature review of both print and web-based sources, from Australia and Overseas. This will ensure that a wide variety of information is obtained, meaning the method will be flexible enough to be used in various circumstances, while still being relevant to Southern (Toowoomba) district.

2) Develop a systematic method for the forensic investigation of pavement failure, which is applicable to the Toowoomba District.

After the literature review is completed, all of information obtained must be 'unified' into a method that is relevant to Toowoomba district. To ensure method is relevant, feedback from local Main Roads staff will be taken into account when developing the method.

3) Select, in conjunction with Main Roads staff, a number of failing pavements within the Toowoomba District. Initial investigation will be on about 8 sites.

Negotiation will help to ensure that best sites are selected for the investigation. Selecting sites that are already due to be tested will help ensure any delays/problems with testing or cost are kept to a minimum.

4) Apply the method to the selected sites.

This will be completed as required by the method developed in step 2. Application may be in the form of either desktop review, visual site examination or experimental/lab testing. Main Roads personnel will probably carry out any testing.

5) Analyse the information obtained from each site and determine if adjustment to the original method is needed.

The results found from applying the method to the sites will be examined, and in consultation with Main Roads staff, any modifications to the method will be listed (may also occur part-way through application if required).

6) Determine if the forensic method developed is appropriate for use in the Toowoomba District, from both a technical viewpoint and an economic viewpoint, and its applicability to the future.

This will be done largely by reviewing the results obtained from steps 4 and 5 above, and will also be determined in consultation with Main Roads personnel. It is anticipated that the method should be flexible enough that the method can be used in varying forms and detail, as required by the size and importance of the project being examined.

7) Report findings through oral presentation at the Project Conference, and in the required written format.

This project will be presented as oral presentation at the Project Conference, and will also be completed in the required written format (electronically as a PDF file).

Timelines

This is best displayed in tabular format as shown below (dates are only approximate). It should be noted that there is overlap between some of the activities.

Table C.1 – Project timelines

No.	Task Name	Start Date	Finish Date	Critical Preceding Items	Details
1	Background	Mar	May	-	First task (independent)
2	Develop Method	Apr	July	1	Requires 1
3	Select sites	Mar	May	-	Independent of steps 1 and 2, critical for 4
4	Apply method to sites	Jun	Sep	2, 3	Requires 2 and 3, critical for 5
5	Analyse results	Jul	Oct	4	Ongoing process
6	Conclusions	Aug	Oct	5	Ongoing process
7	Report on Findings	Sep	Oct	6	Final stage: requires all others complete

Appendix D

Safety Issues and Resource Planning

Safety Issues

This is simply done in table form, with all hazards and risks being identified. Due to the nature of the project, the only stage that poses a risk is step 4 where the method is applied to the selected test sites.

Table D.1 – Possible safety issues in project

Task	Investigation & Testing of Sites	Investigation & Testing of Sites
Hazard	Failure of equipment being used	Struck by passing vehicle
Likelihood	Very Slight	Slight
Exposure	Occasionally - Frequently	Occasionally - Frequently
Consequences	Equipment damage, injury, death	Injury, death
Risk Control	Equipment in good working order Proper operating procedure Experienced personnel	Use traffic control if required Use caution at all times Wear Personal Protection Equipment

The hazard likelihood is difficult to quantify, but should not me increased by project relative to ordinary tasks in similar area. The amount of exposure varies depending on amount of work being carried out. However, since the method will utilise existing testing procedures, exposure should not be increased.

The risk control method for both would include all measures required using Workplace Health and Safety Act. Since testing will probably be carried out by Main Roads personnel, risk control for both hazards will almost certainly have already been considered.

Resource Planning

Due to the nature of the project, the resources required are minimal, and there are not any critical resources that are likely to cause problems. The project could continue even if Main roads documents, staff or testing personnel and equipment were not available, since the main focus of the project is on developing a method. The resources likely to be required are shown in the table below.

Table D.2 – Resources required for project

Resource	**Source**	**Possible Problems**	**Costs**	**Other Details**
Computer: Word Processing, Internet Access	USQ, Main Roads	Nil	Nil	Unlikely to cause any delay in project
USQ Library	USQ	Almost Nil	Nil	Unlikely to cause any delay in project
Main Roads documents	Main Roads	Not able to be found, already being used	Nil	Possibly some delay, but can be managed by planning
Main Roads staff	Main Roads	Too busy to help	Nil	Manage by planning meeting times
Testing Personnel and Equipment	Main Roads, RoadTek	May not be available when required	Yes	Best to use sites that have already been tested, or are due to be tested
Car	Main Roads, Personal	Not able to be used	Nil	Unlikely to cause any delay in project

Appendix E

Bowenville-Dalby: Test Results

Trenching Part 1: 21/10/2003

Taken from *Investigation of Failures on Job 67/18B/302.1* (2004)

Lab no. 03/....	1227	1228	1230	1231	1232
Chainage	75552	75552	75552	75552	75552
Sample Depth (mm)	0-150	0-150	0-150	0-150	0-150
℄ OFFSET (m)	@2.2	@3.8	@2.9	@2.2	@3.6
Test Results	W/B	W/B	W/B	E/B	E/B
Particle Size Distribution					
Percent Passing					
A.S. Sieve Size (mm)					
75.0					
53.0					
37.5	100	100			
19.0	98	99			
9.5	64	64			
4.75	47	49			
2,36	31	34			
0,425	17	18			
0.075	11	11			
Liquid Limit	24.2	22.8			
Plastic Index	8.4	6.2			
Linear Shrinkage	5.6	4.6			
P.I. * % < 0.425 mm	142	112			
L.S. * % < 0.425 mm	95	83			
Ratio 0.075/0.425 mm	0.67	0.61			
Insitu Moisture (%)	3.8	4.3	4.6	3.1	3.3

Figure E.1 – Summary of Test Results: Bowenville-Dalby – 21/10/2003

Trenching Part 2: 27/11/2003

Taken from *Investigation of Failures on Job 67/18B/302.1* (2004)

Lab no. 03/....	1294	1295	1296	1297	1298	1299	1300	1301	1302	1303	1304	1305	1306	1307
Chainage	75553	75553	75553	75553	75553	75553	75551	75551	75551	75551	75551	75551	75551	75551
Sample Depth (mm)	0-50	100-150	250-300	0-50	100-150	250-300	0-50	100-150	250-300	200-250	0-50	100-150	250-300	200-250
℄ OFFSET (m)	@3.8	@3.8	@3.8	@2.2	@2.2	@2.2	@3.8	@3.8	@3.8	@3.8	@2.2	@2.2	@2.2	@2.2
Test Results	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B	W/B
Particle Size Distribution														
Percent Passing														
A.S. Sieve Size (mm)														
75.0														
53.0														
37.5	100			100			100				100			
19.0	99			98			99				99			
9.5	65			71			67				66			
4.75	45			54			48				51			
2.36	29			37			32				36			
0.425	17			19			18				18			
0.075	13			12			13				11			
Liquid Limit	27.4			23.8			27.2			24.0	23.8			22.4
Plastic Index	13.0			8.2			12.4				8.0			
Linear Shrinkage	7.2			4.6			7.8			4.6	4.8			4.2
P.I. * % < 0.425 mm	224			157			228				145			
L.S. * % < 0.425 mm	124			88			144				87			
Ratio 0.075/0.425 mm	0.73			0.62			0.73				0.62			
Insitu Moisture (%)	5.5	4.7	4.9	4.1	3.9	4.4	5.3	5.0	3.7	3.5	3.8	4.1	4.0	3.0

Figure E.2 – Summary of Test Results: Bowenville-Dalby – 27/11/2003

Appendix F

Gatton Bypass: PAVDEF Deflection Testing Results

Details

A PAVDEF Deflection Survey was carried out along the newly constructed eastbound lanes of the Gatton Bypass, Warrego Highway (18A) in November 2003. The full data is contained in the report *PR 2254D PAVDEF Deflection Survey: Gatton Bypass Duplication* (Queensland Department of Main Roads, 2003).

Landmarks recorded during the run are shown in the table. Initially the data was recorded over 50m lengths for both the inner and outer wheelpaths of both the inner and outer lanes. Data recorded was the 90% highest deflection in mm, mean curvature function in mm, 10% lowest subgrade CBR, and the 10% lowest deflection ratio.

In the following graphs, the results for the outer wheelpath of the outer lane and the inner wheelpath of the inner lane are shown, since they represent the extremes of the data. Average trendlines for every 10 data points are shown. For conversion purposes, the road chainage is equal to the job chainage + 30.36 (in km). Unless stated otherwise, the chainages used for the graphs are the job chainages.

Table F.1 – Landmarks during Gatton Bypass PAVDEF Deflection Survey

Job Chainage (m)	Details
25110	Start Gatton-Esk Road
25400	Eastern Overpass
26090	Bridge at Lockyer Creek
28920	Adare Road Bridge
32230	Smithfield Bridge
36360	Philp Bridge
39350	Bridge at Sandy Creek
44660	Helidon Overpass
45200	End of Run

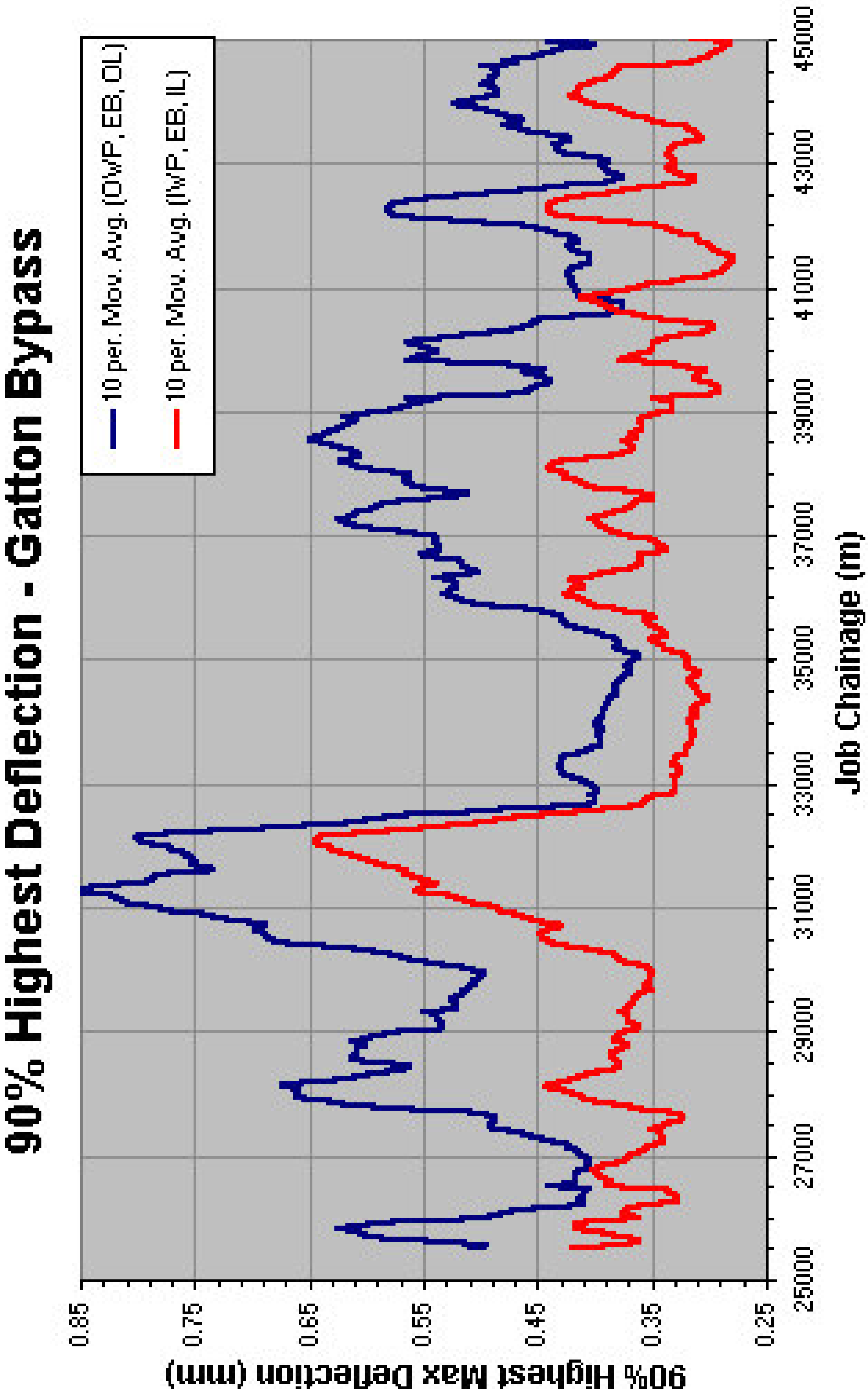


Figure F.1 – 90% Highest Maximum Deflection for eastbound lanes of Gatton Bypass

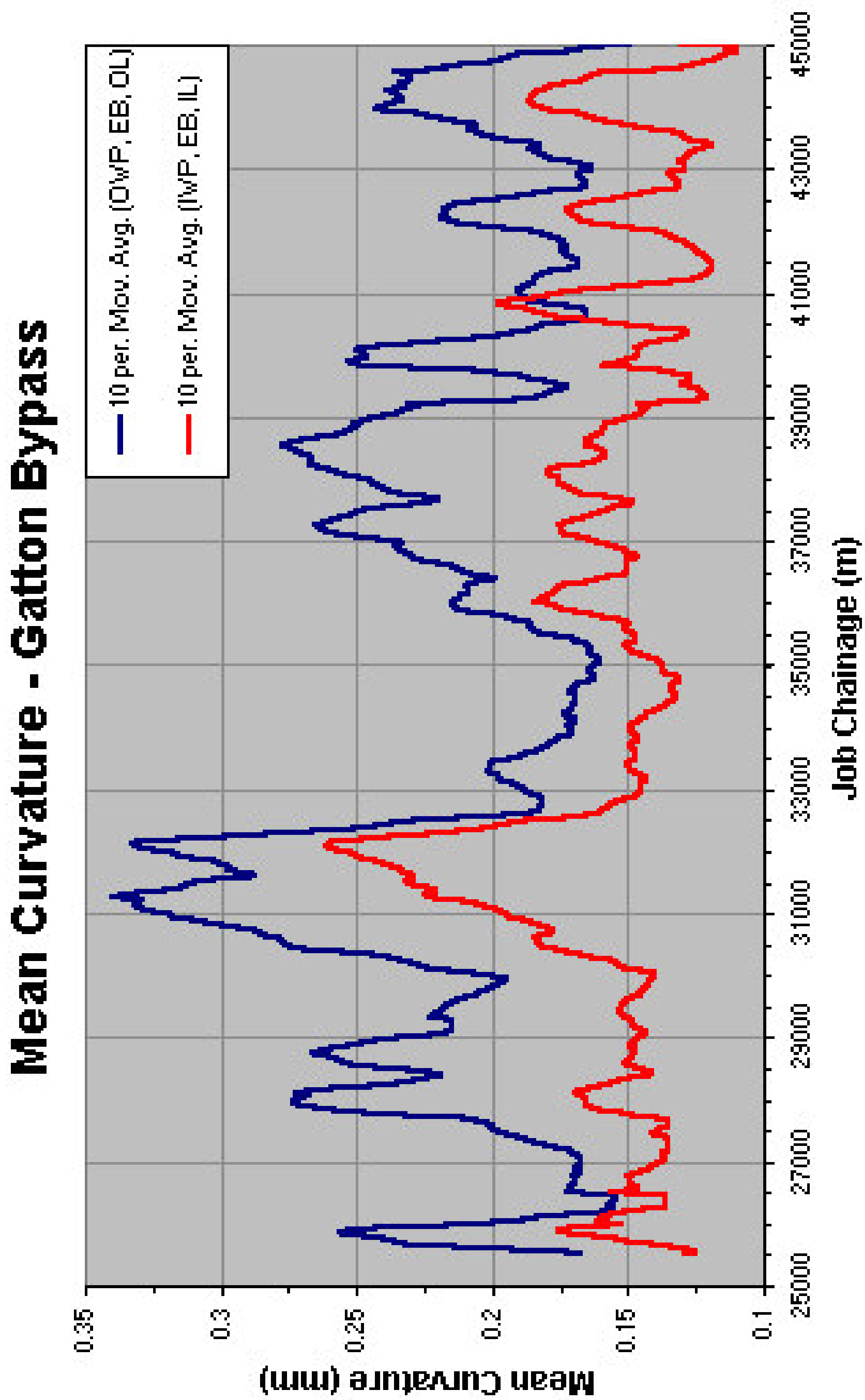


Figure F.2 – Mean Curvature for eastbound lanes of Gatton Bypass

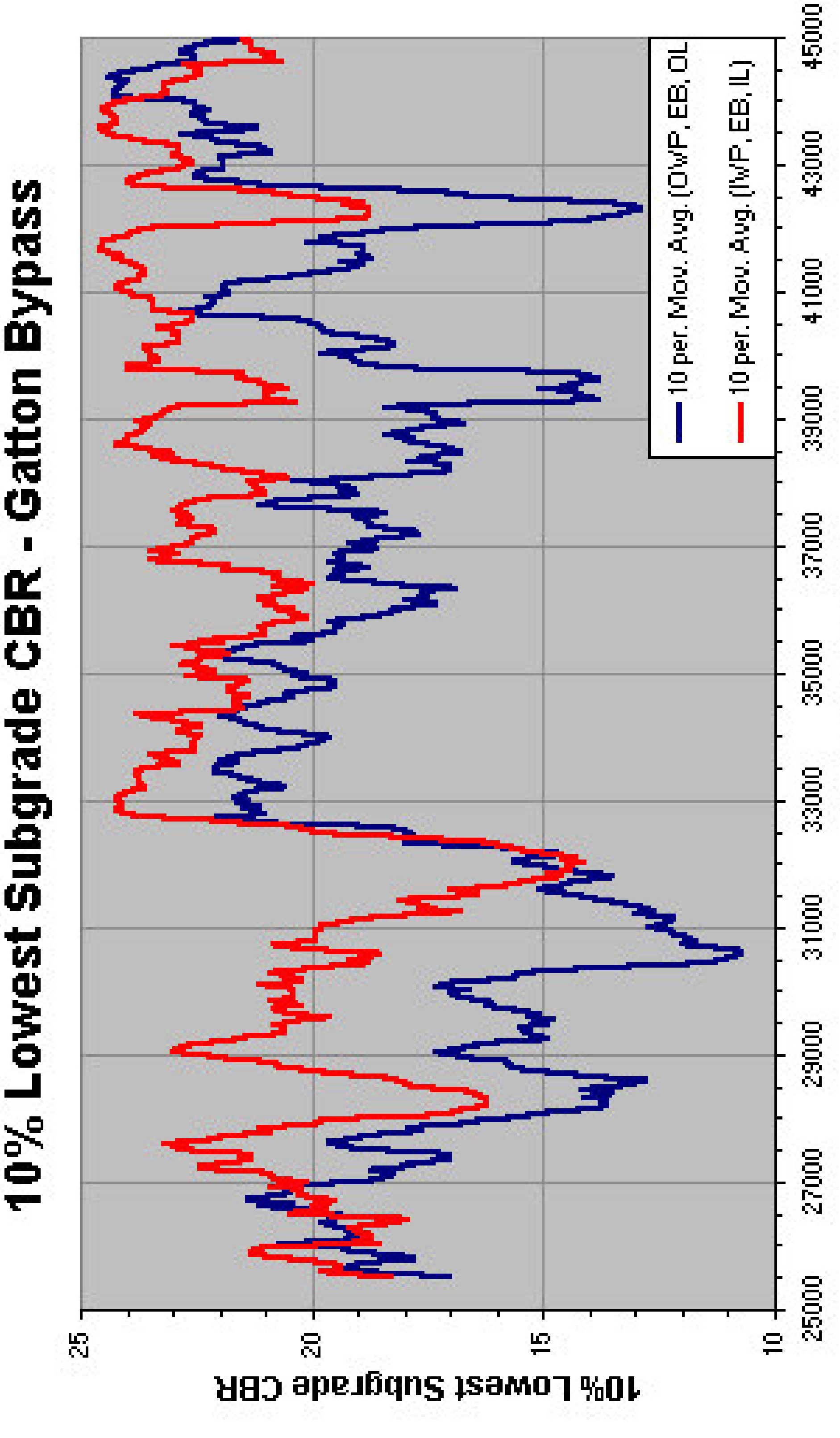


Figure F.3 – 10% Lowest Subgrade CBR for eastbound lanes of Gatton Bypass

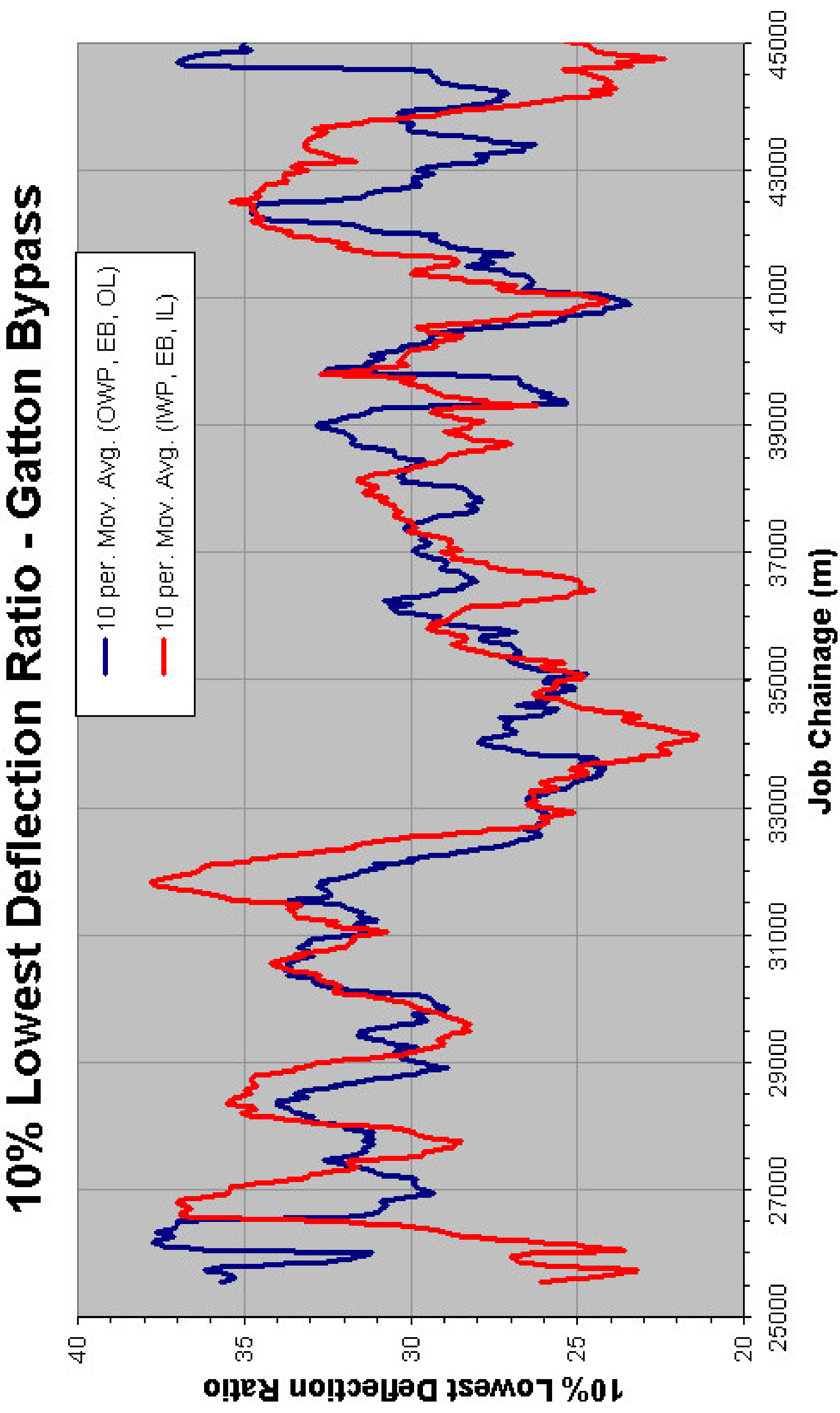


Figure F.4 – 10% Lowest Deflection Ratio for eastbound lanes of Gatton Bypass

Appendix G

Gatton Bypass: FWD Deflection Testing Results

Details

A FWD Deflection Survey was carried out along the newly constructed eastbound lanes of the Gatton Bypass, Warrego Highway (18A) in November - December 2003 at 19 sites, over a total length of 3.47 km.

The results were summarised in a report, *Gatton Bypass Duplication - Analysis of FWD Deflection Results* (Queensland Department of Main Roads, 2004c) prepared in February 2004, by the Pavements, Materials and Geotechnical Division, Road Systems and Engineering Group.

Since the results for deflection, curvature and deflection ratio were similar to those found using PAVDEF, these graphs will not be shown. Instead, the graphs show the results from back calculation of the layer moduli using the FWD deflection data, for the outer wheelpath of the inner and outer lanes. This analysis was completed for three defined layers of the base material, with the data being averaged over every 10 data points.

The design moduli are the expected values for the material. For 0 – 133 mm depth the design moduli was 350 MPa, for 133 - 266 mm depth, the design moduli was 290 MPa, and for 266 – 400 mm depth, the design moduli was 230 MPa. These figures are reproduced on the graphs as solid black lines.

For conversion purposes, the road chainage is equal to the job chainage + 30.36. Unless stated otherwise, the chainages used for the graphs are the job chainages.

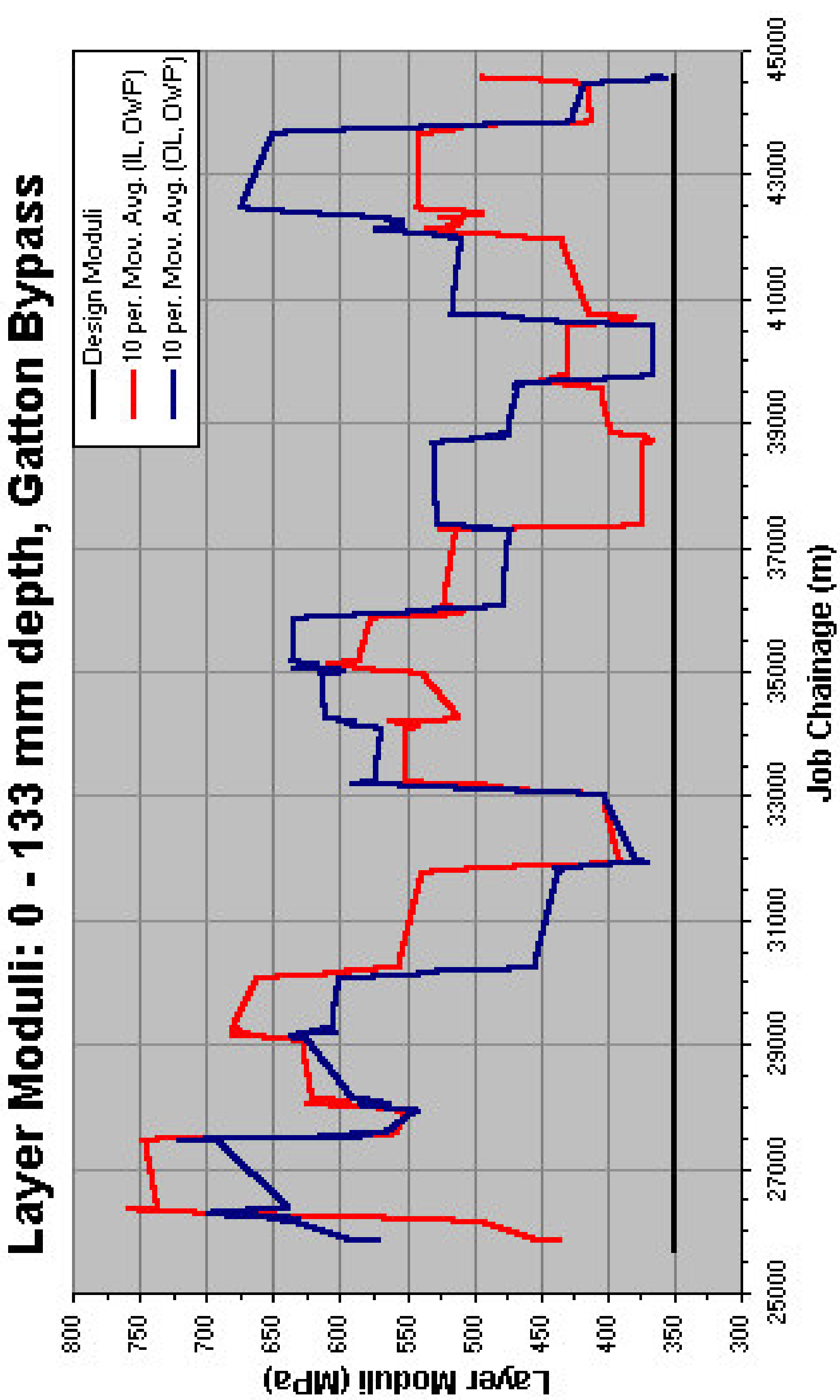


Figure G.1 – Back calculated layer moduli for 0 – 133 mm depth of base material – Gatton Bypass

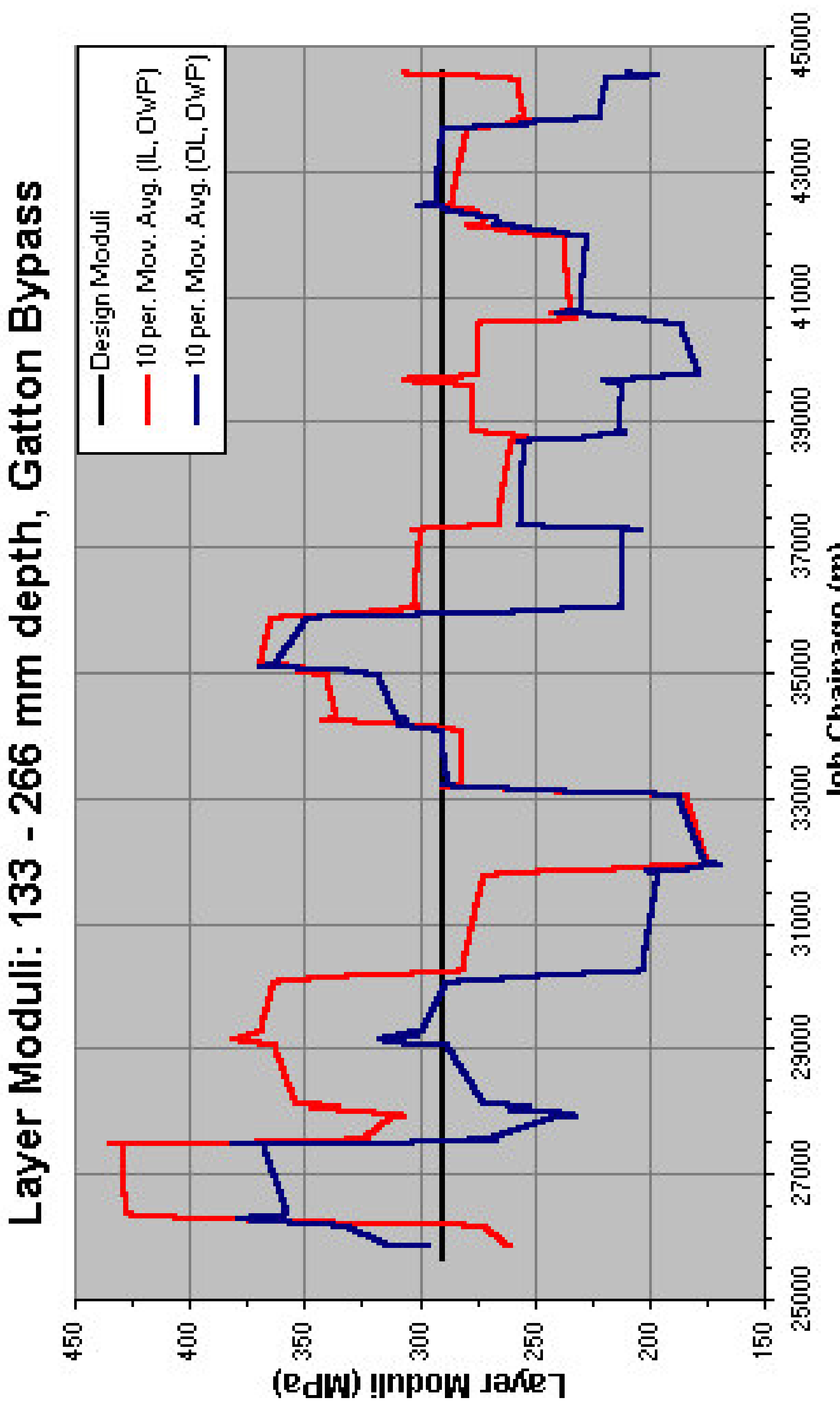


Figure G.2 – Back calculated layer moduli for 133 – 266 mm depth of base material – Gatton Bypass

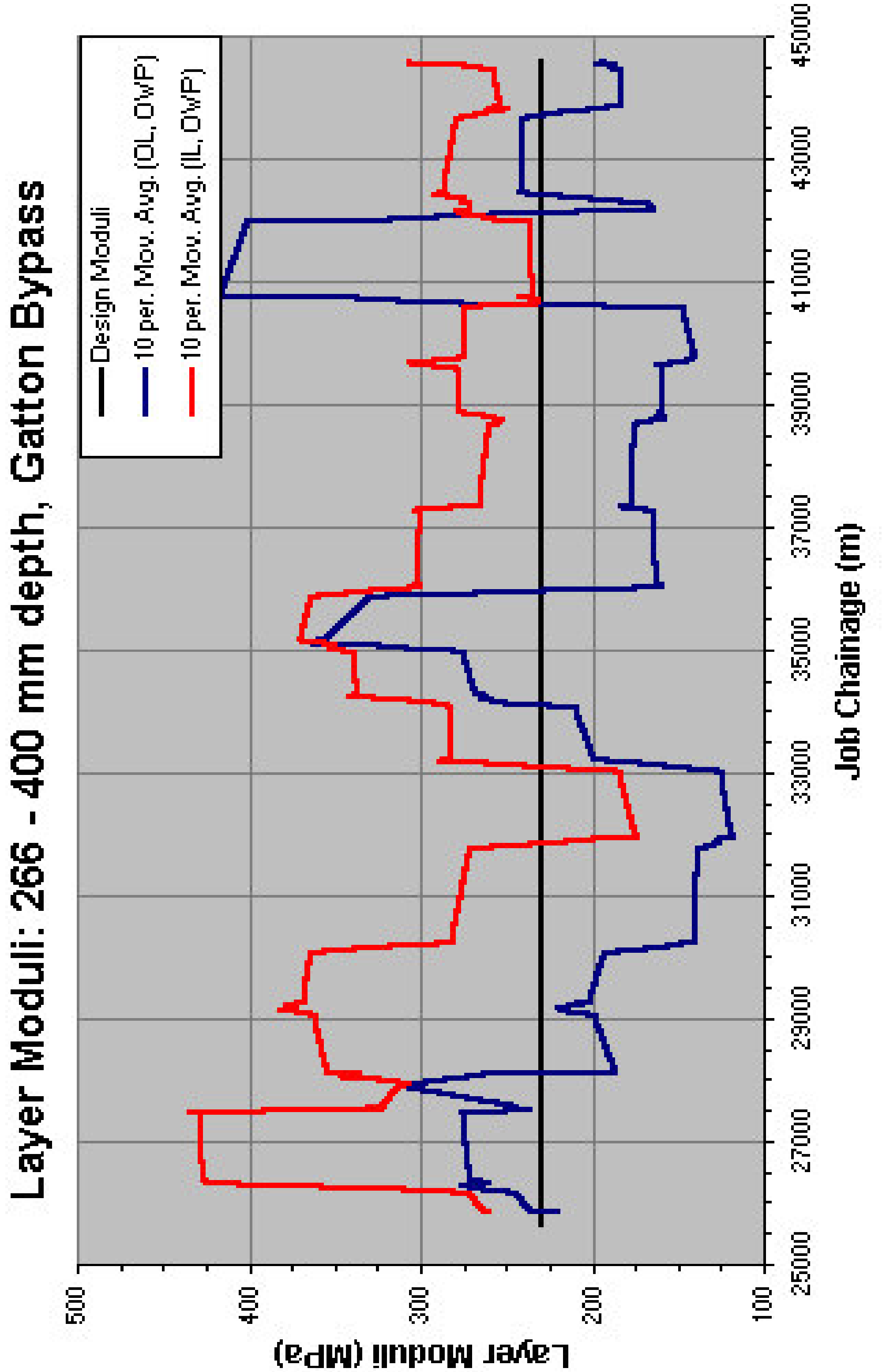


Figure G.3 – Back calculated layer moduli for 266 – 400 mm depth of base material – Gatton Bypass

Appendix H

Warrego Highway through Toowoomba: PAVDEF Deflection Testing Results

Details

A PAVDEF Deflection Survey was conducted along the all lanes, both eastbound and westbound, of the Warrego Highway through Toowoomba in May 2004. The chainages tested were 18A: 92.32 – 95.0 km, and 18B: 0.0 – 8.10 km. The full data is contained in the report PR *2331A PAVDEF Deflection Survey: Warrego Highway through Toowoomba* (Queensland Department of Main Roads, 2004b).

Landmarks recorded during the run are shown in the table. Initially the data was recorded over 100m lots, in both wheelpaths of both lanes of the eastbound and westbound directions. Data recorded was the 90% highest deflection in mm, mean curvature function in mm, 10% lowest subgrade CBR, and the 10% lowest deflection ratio.

Table H.1 – Landmarks during Warrego Highway PAVDEF Deflection Survey – 18A

Chainage (m)		
Road	**Job**	**Details**
92320	20	Intersection with Herries Street
92750	450	James Street, PRP 28
93210	910	Intersection with MacKenzie Street
93750	1450	Intersection with Kitchener Street, PRP 29
94570	2270	Intersection with Hume Street, PRP 30
94790	2490	Intersection with Neil Street
94990	2690	Stop: Intersection with Ruthven Street (PRP 31A)

Table H.2 – Landmarks during Warrego Highway PAVDEF Deflection Survey – 18B

Chainage (m)		
Road	**Job**	**Details**
0	2700	Intersection with Ruthven Street, PRP 1
330	3030	Rail Crossing
490	3190	Intersection with Prescott Street
1000	3700	Intersection with West Street, PRP 2
1090	3790	Seal change
2240	4940	Intersection with Anzac Avenue, PRP 3
2420	5120	Intersection with Karpool Street
2920	5620	Intersection with Hursley Road, PRP 4
3750	6450	Intersection with Taylor Street, PRP 5
4510	7210	Bridge Street, PRP 6
5280	7980	Intersection with Richmond Drive
5770	8470	Intersection with Greenwattle Street
6620	9320	Intersection with McDougall Street, PRP 7
7290	9990	Intersection with Boundary Road
7300	10000	Shire Boundary, PRP 8
8080	10780	End, Nugent pinch Road

In the following graphs, the results for the outer wheelpath of the outer lane and the inner wheelpath of the inner lane are shown, since they represent the extremes of the data. Average trendlines for every 5 data points are shown. To enable easier graphing and summarisation of the data, a job chainage was created, where for 18A, job chainage = road chainage – 92.3, and for 18B, job chainage = road chainage + 2.7 (all in km).

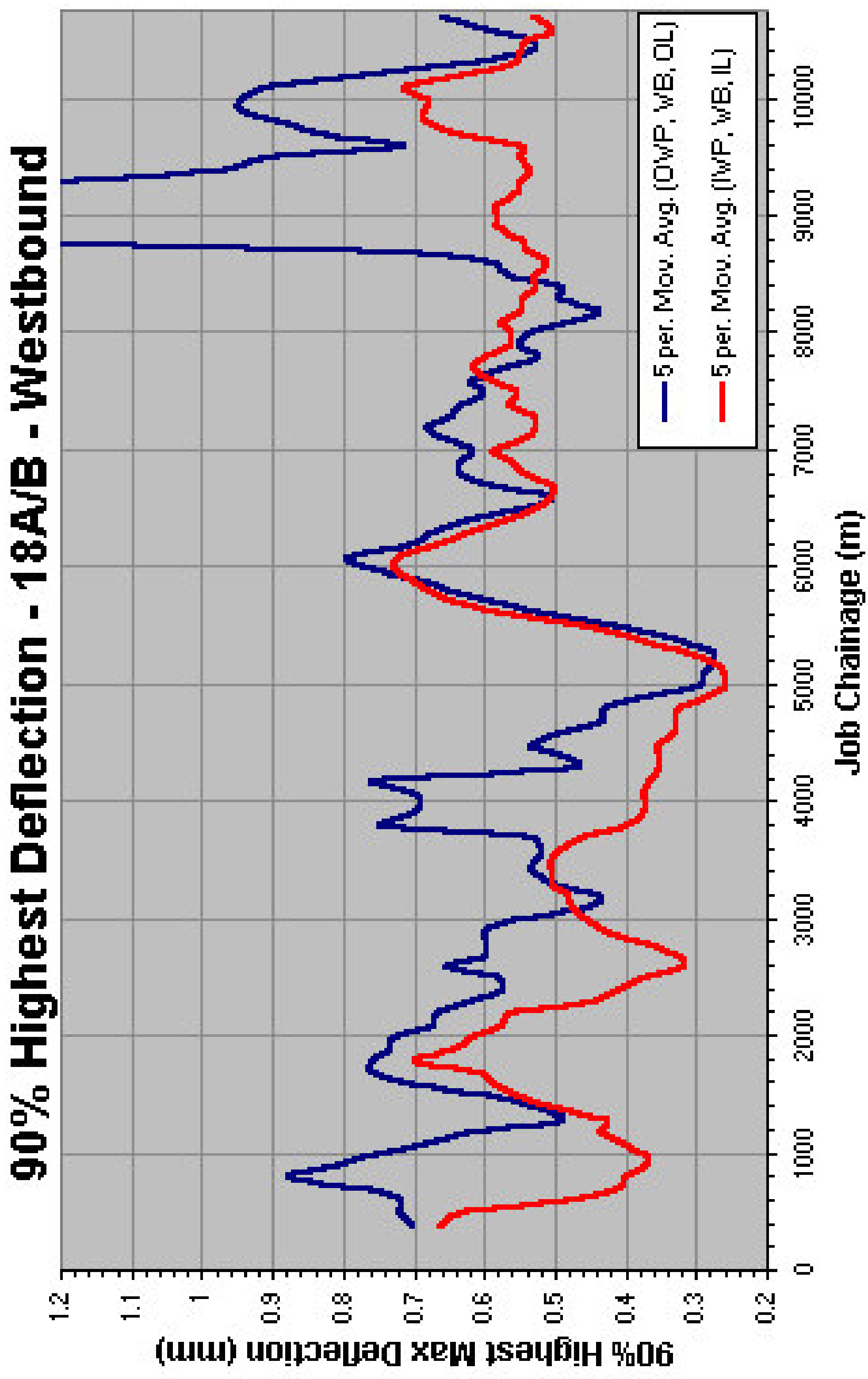


Figure H.1 – 90% Highest Maximum Deflection for westbound lanes of Warrego Highway through Toowoomba

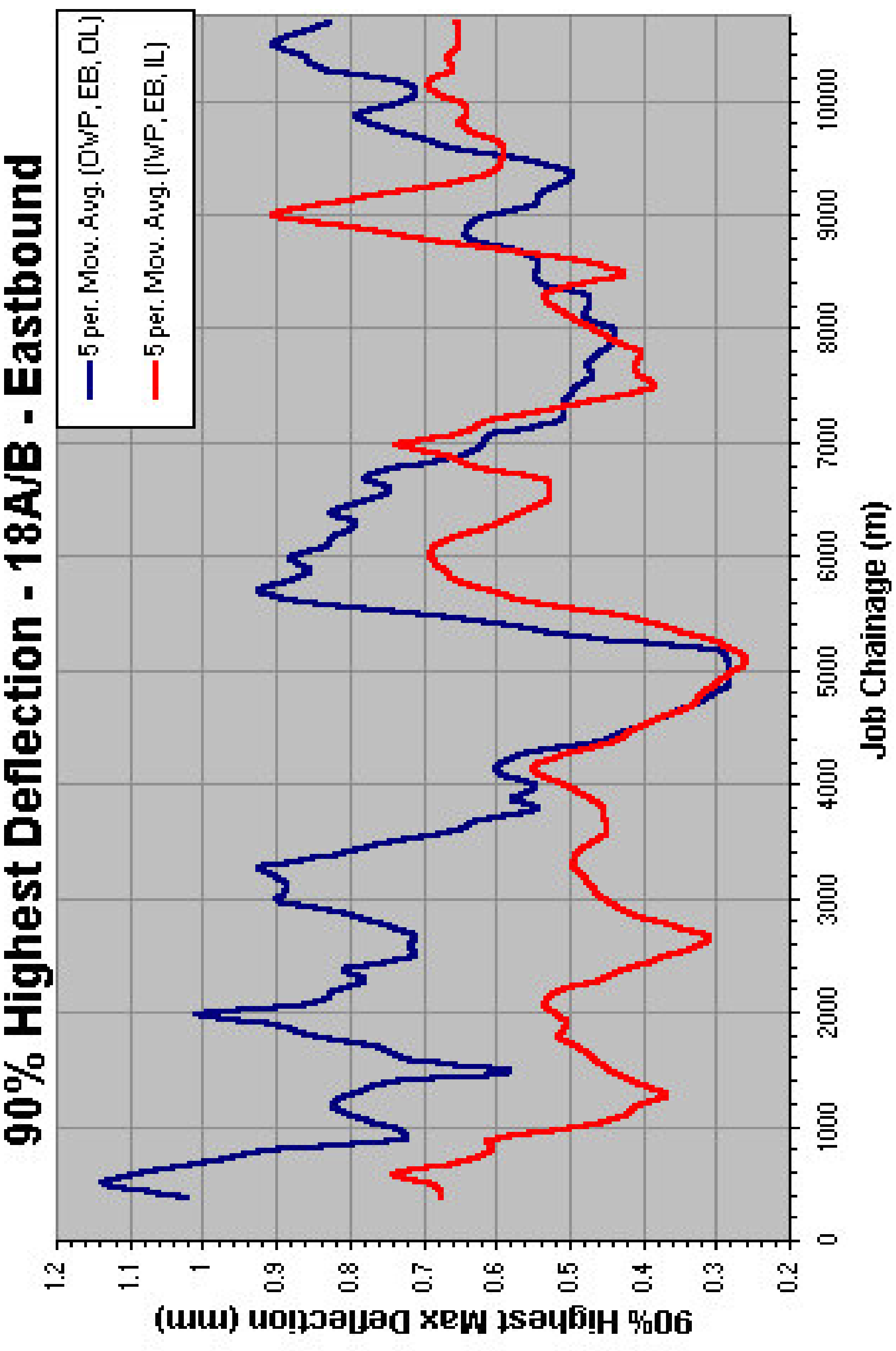


Figure H.2 – 90% Highest Maximum Deflection for eastbound lanes of Warrego Highway through Toowoomba

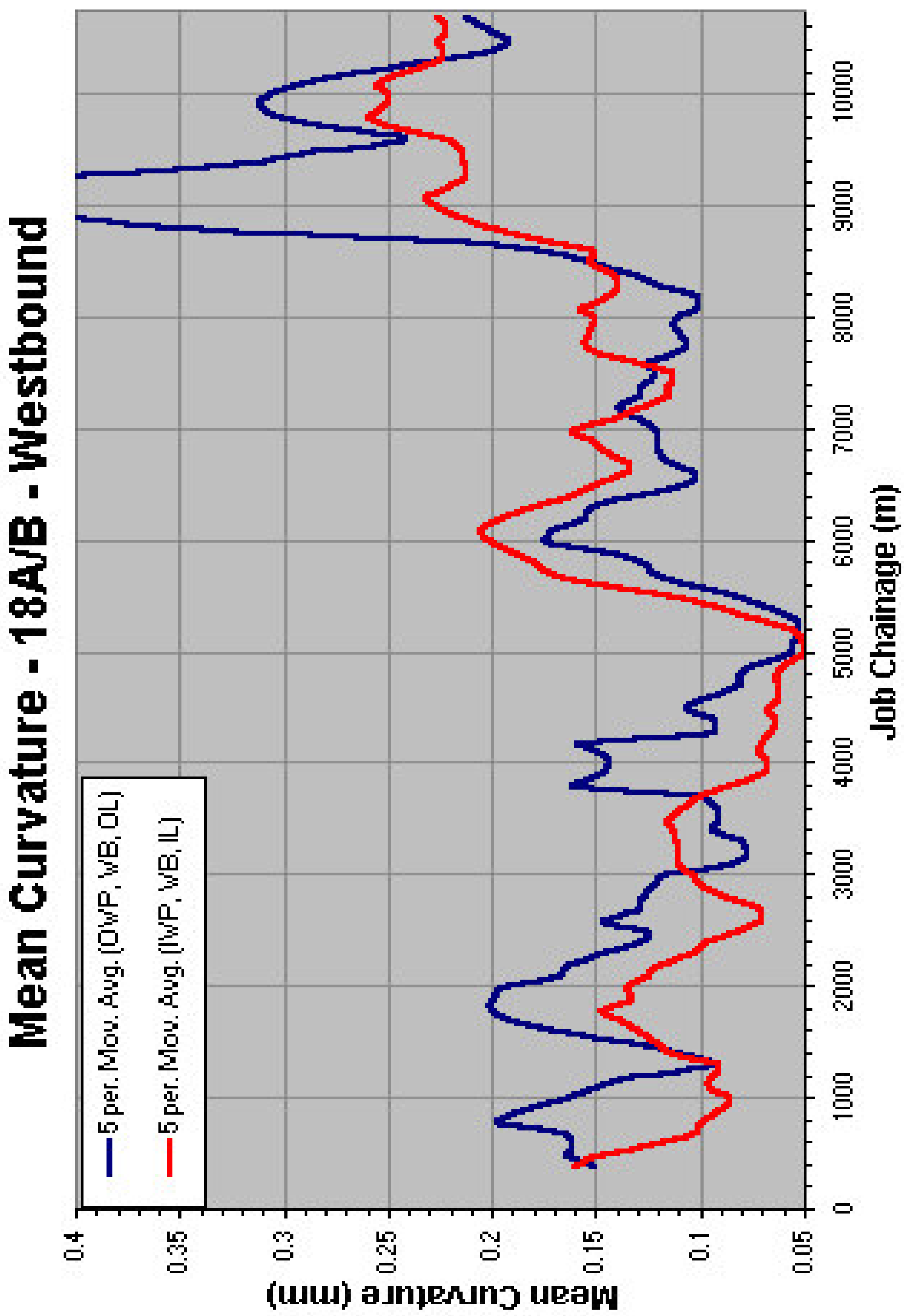


Figure H.3 – Mean Curvature for westbound lanes of Warrego Highway through Toowoomba

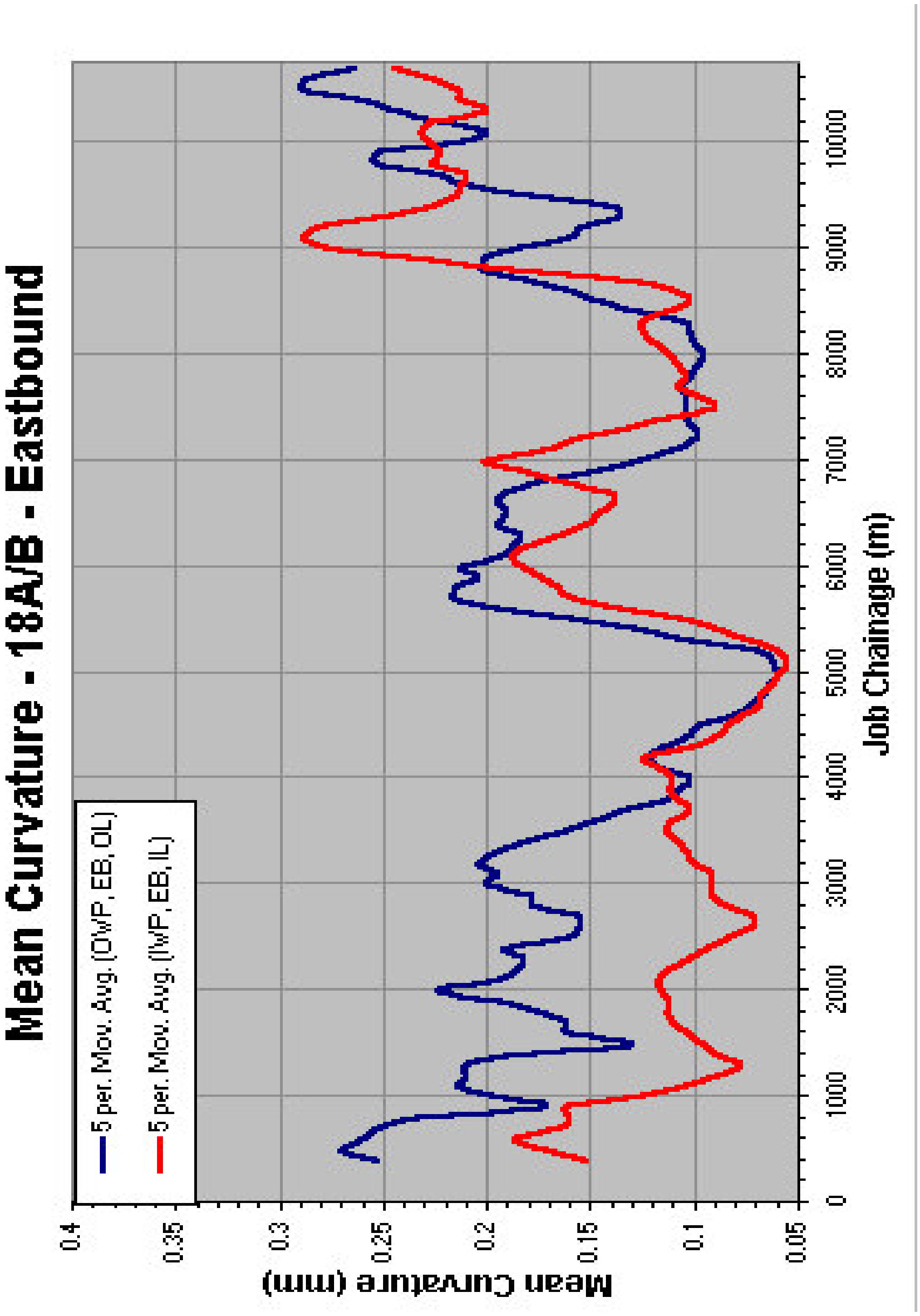


Figure H.4 – Mean Curvature for eastbound lanes of Warrego Highway through Toowoomba

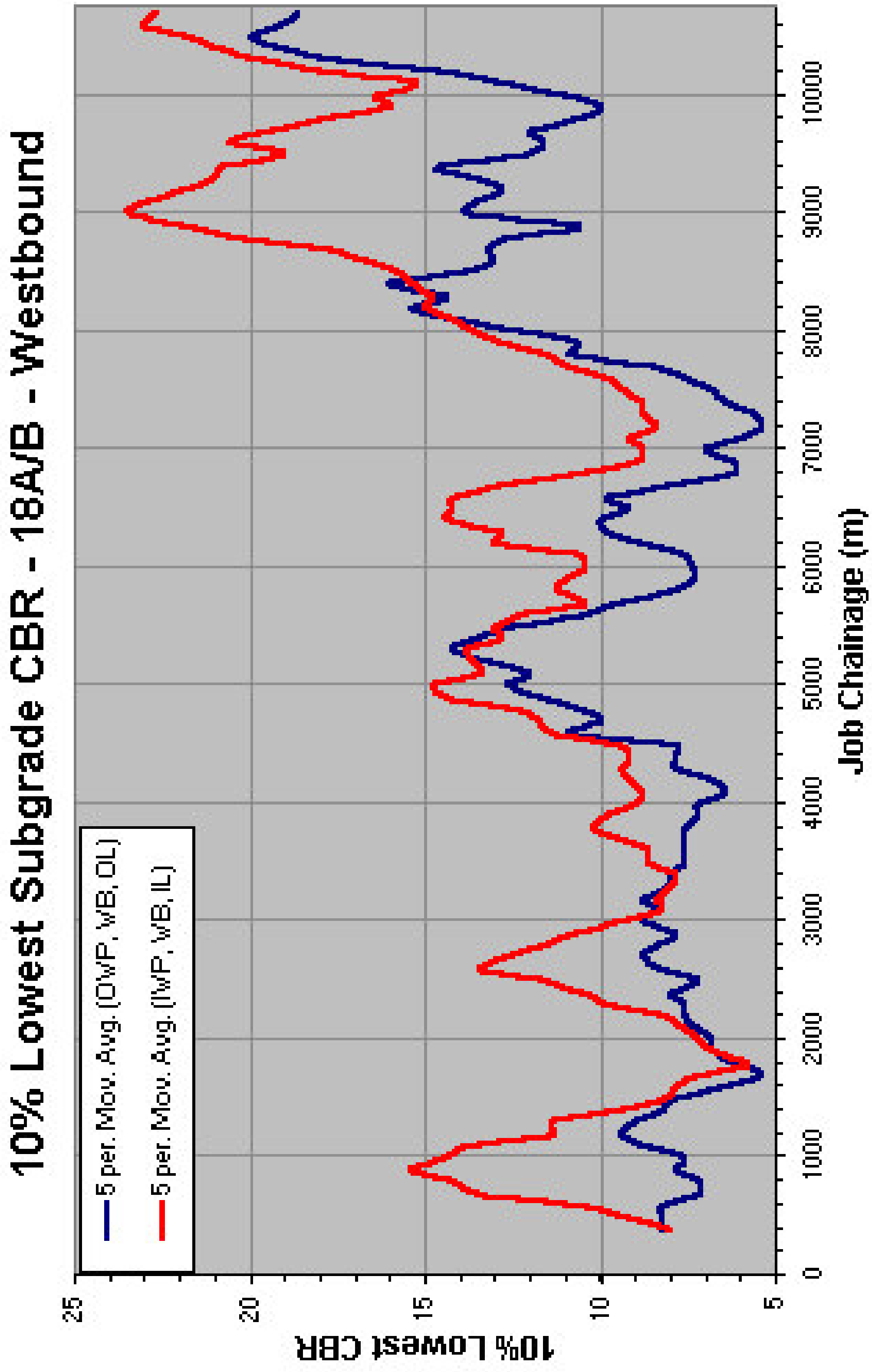


Figure H.5 – 10% Lowest Subgrade CBR for westbound lanes of Warrego Highway through Toowoomba

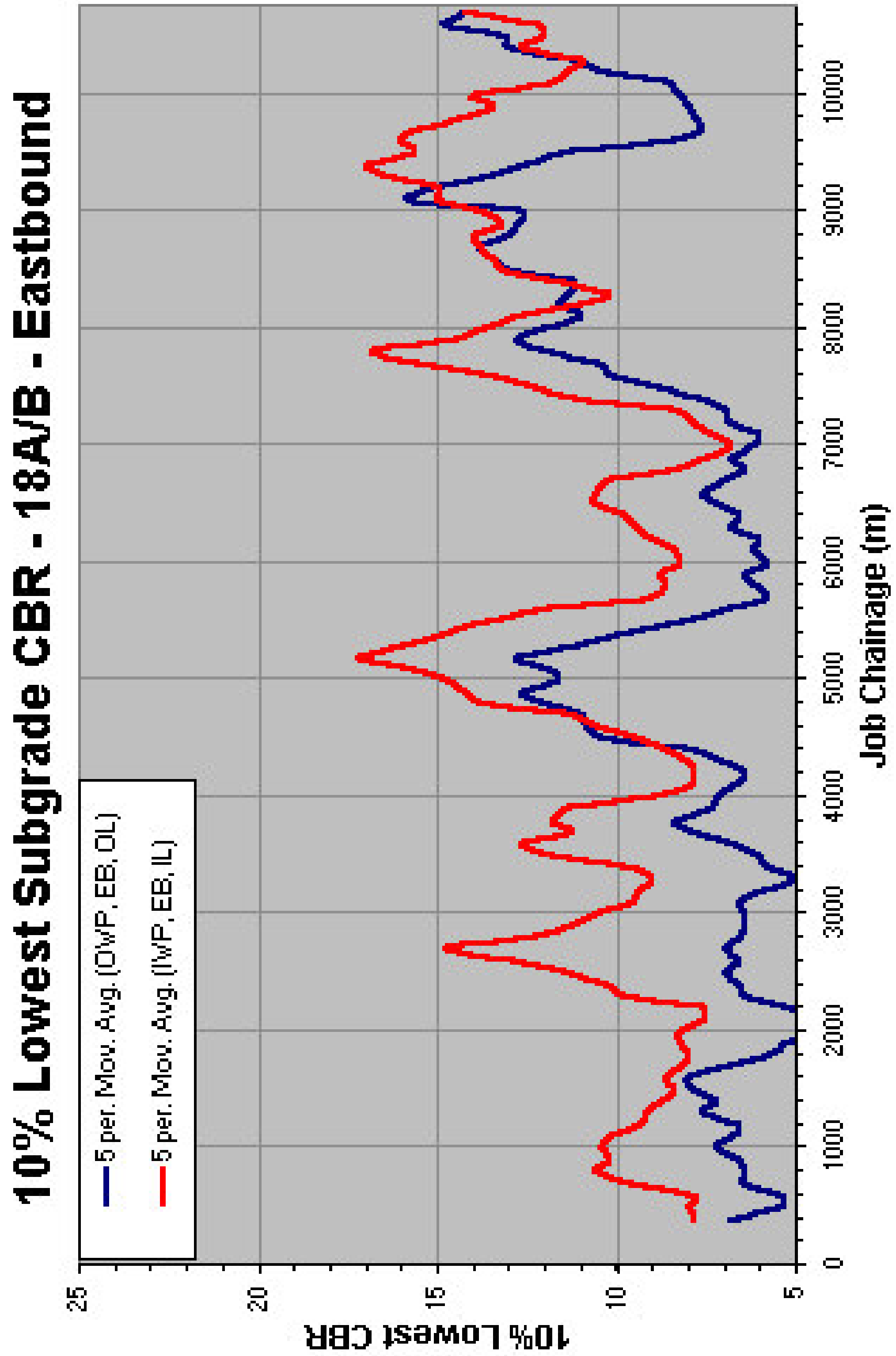


Figure H.6 – 10% Lowest Subgrade CBR for eastbound lanes of Warrego Highway through Toowoomba

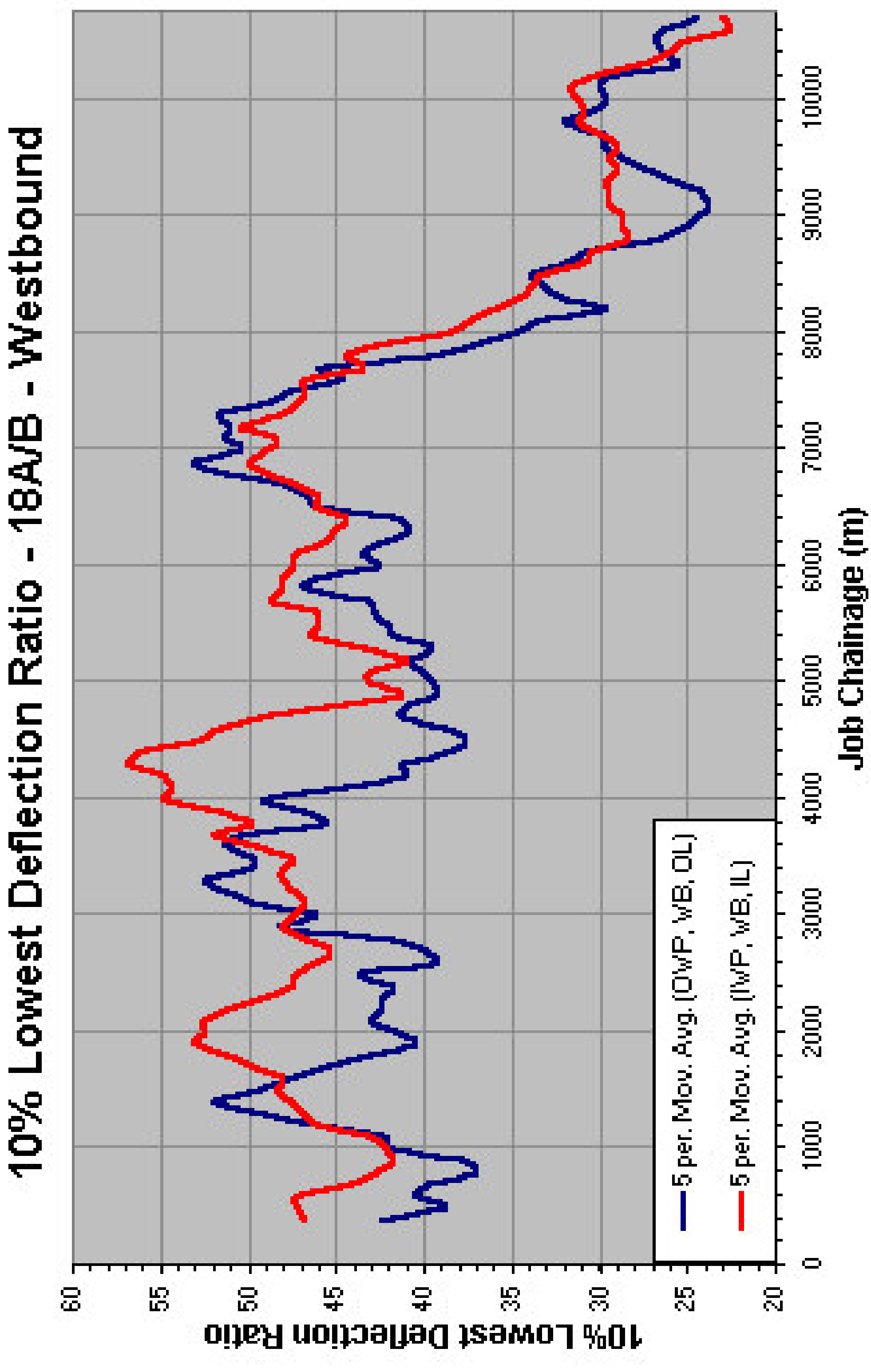


Figure H.7 – 10% Lowest Deflection Ratio for westbound lanes of Warrego Highway through Toowoomba

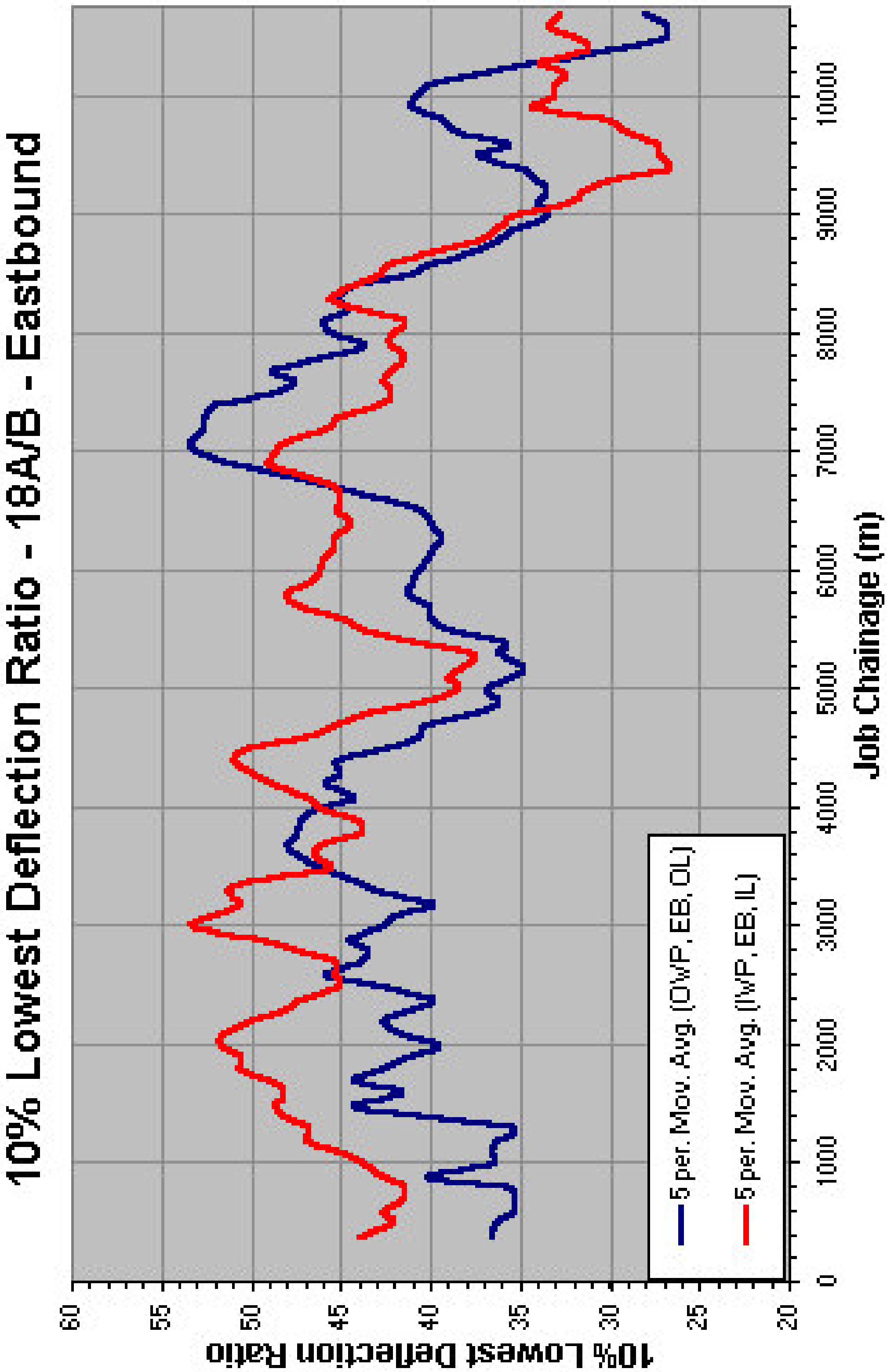


Figure H.8 – 10% Lowest Deflection Ratio for eastbound lanes of Warrego Highway through Toowoomba

Appendix I

Yaralla Deviation: PAVDEF Deflection Testing Results

Details

Deflection testing along the Yaralla Deviation using the PAVDEF machine was conducted on two separate occasions. The first run was conducted in December 2001, close to the end of construction in the westbound direction only, from chainage 5.6 – 23.2 km. The second run was conducted in July 2004, in both the eastbound and westbound directions, from chainage 5.0 – 23.6 km.

The full data is contained in the reports *PR 2080A PAVDEF Deflection Survey: Yaralla Deviation* (Queensland Department of Main Roads, 2001a) and *PR 2348A PAVDEF Deflection Survey: Yaralla Deviation* (Queensland Department of Main Roads, 2004a).

Landmarks recorded during the run are shown in the table. Initially, the data was recorded over 100m lengths, in both wheelpaths of the lanes that were tested. Data recorded was the 90% highest deflection in mm, mean curvature function in mm, 10% lowest subgrade CBR, and the 10% lowest deflection ratio.

In the following graphs, the results for the outer wheelpath of the westbound lane in 2001 and 2004 are compared. Average trendlines for every 10 data points are shown.

Table I.1 – Landmarks during Yaralla Deviation PAVDEF Deflection Survey

Chainage (m)	Details
5030	Start Job
5580	Intersection with 340 (Dalby-Kogan)
8130	Intersection with Yaralla Road
11020	Intersection with Yaralla Wheat Road
14740	Entrance to Lleumeah Property
16630	Intersection with Gradel Road
18740	Intersection with Luchts Road
20780	Intersection with Pirrinuan Apunyal Road
23230	Intersection with Old Warrego Turnoff

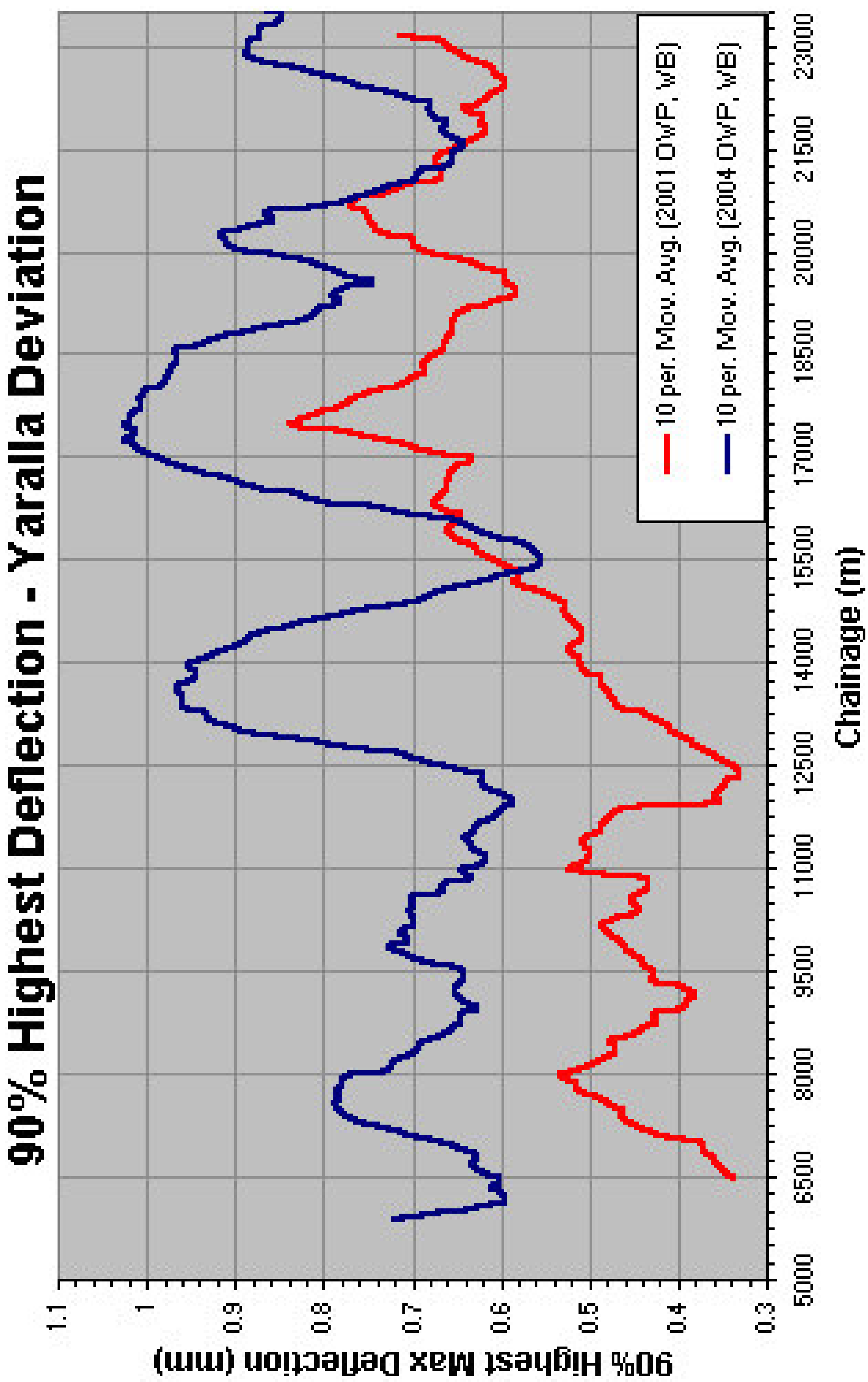


Figure I.1– 90% Highest Maximum Deflection for Yaralla Deviation in 2001 and 2004

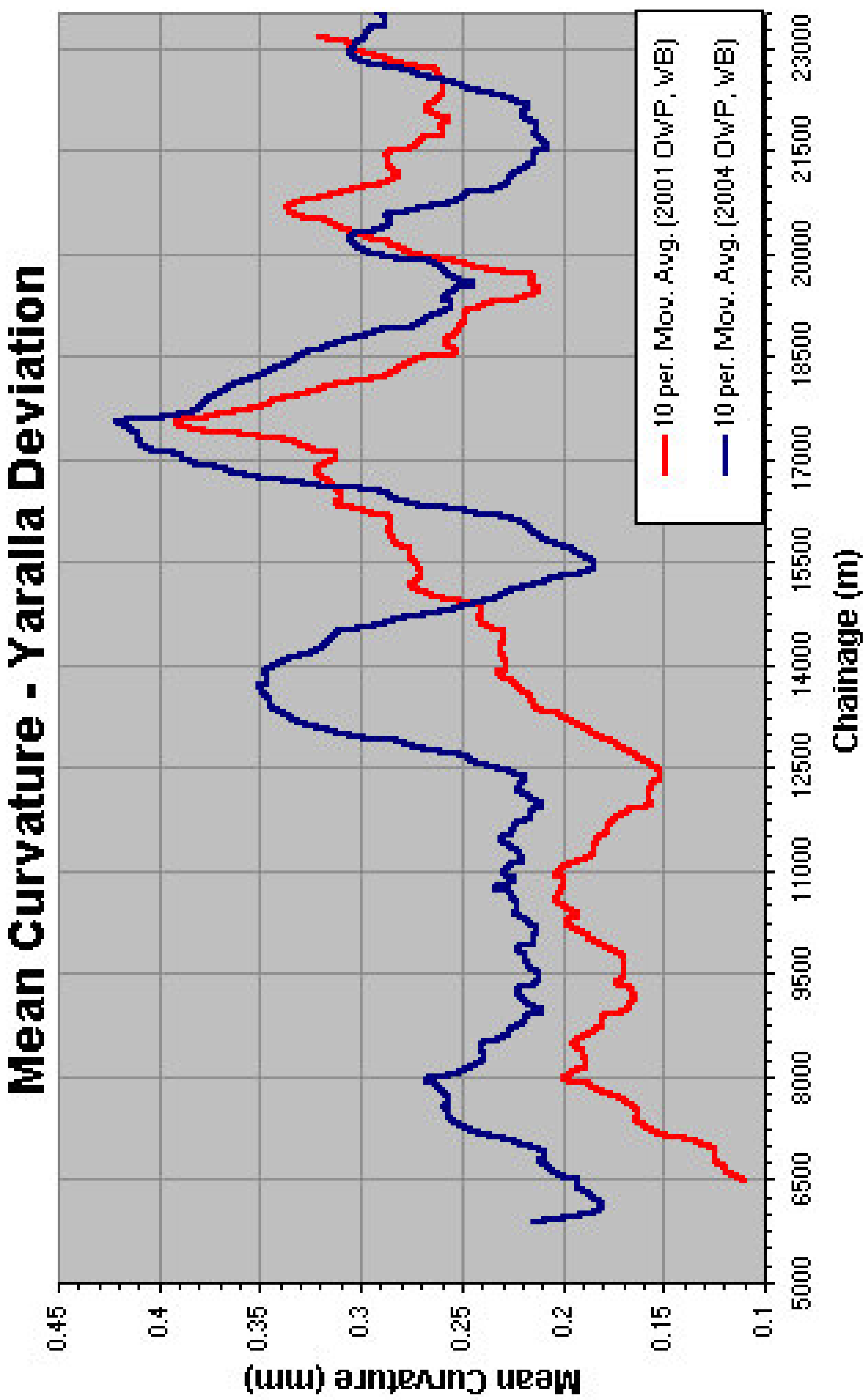


Figure I.2 – Mean Curvature for Yaralla Deviation in 2001 and 2004

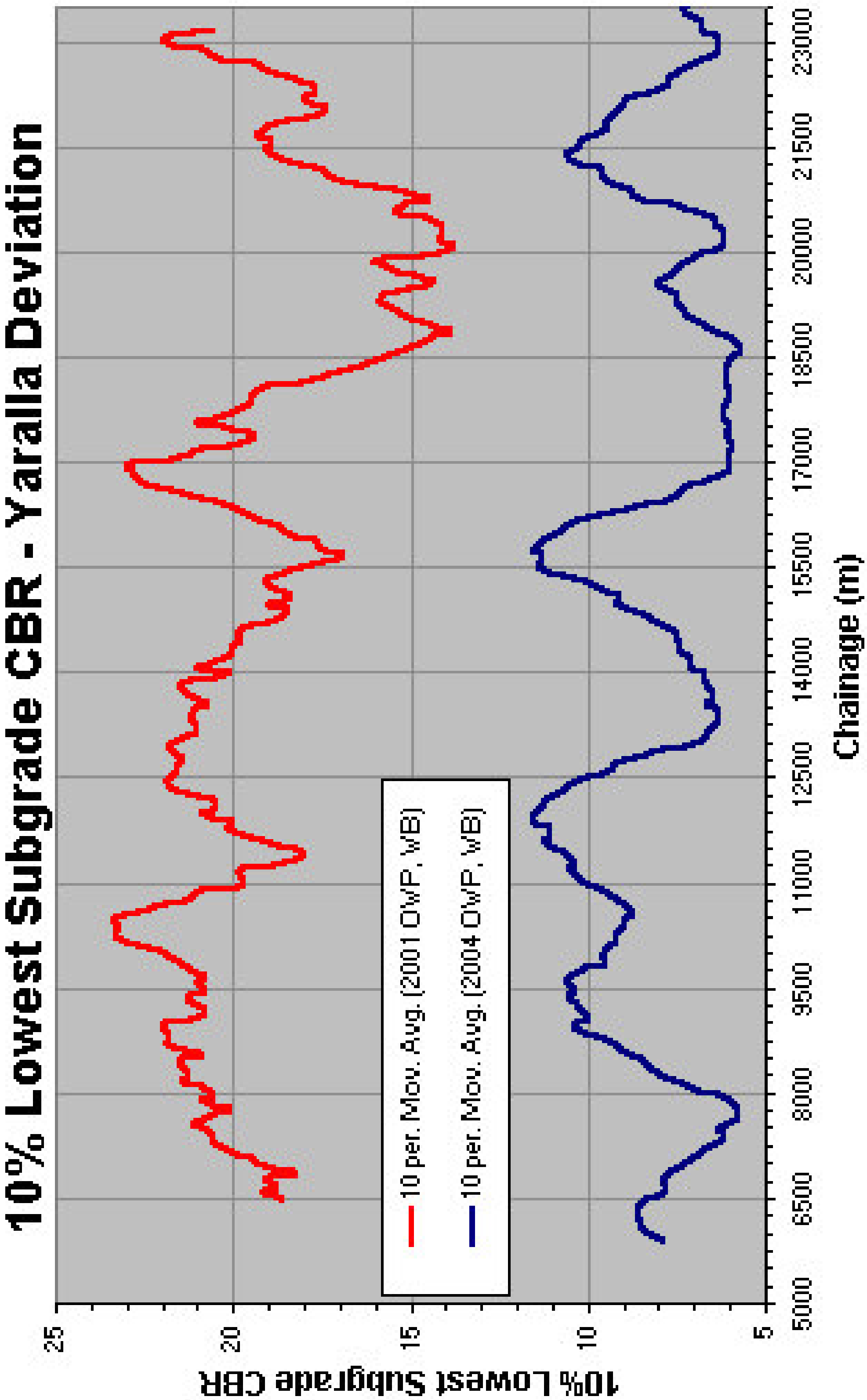


Figure I.3 – 10% Lowest Subgrade CBR for Yaralla Deviation in 2001 and 2004

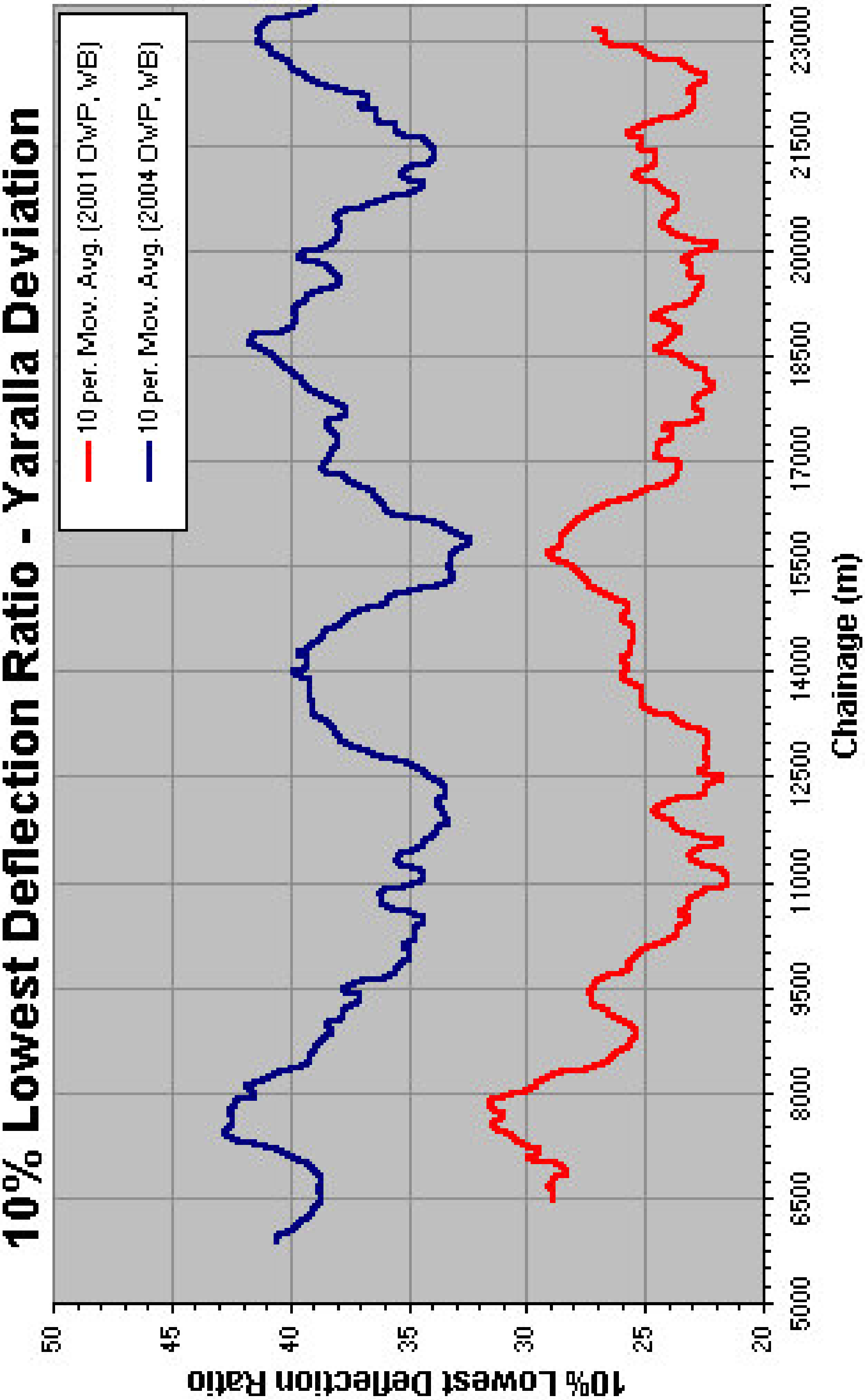


Figure I.4 – 10% Lowest Deflection Ratio for Yaralla Deviation in 2001 and 2004