Pavement deterioration models for asphalt-surfaced pavements in New Zealand

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T. F. P. Henning
Department of Civil and Environmental Engineering, University of Auckland

D. C. Roux
MWH New Zealand Ltd, Auckland

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Private Bag 6995, Wellington 6141, New Zealand
Telephone 64 4 894 5400; facsimile 64 4 894 6100
research@nzta.govt.nz
www.nzta.govt.nz

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Terms and abbreviations

AADT	Annual Average Daily Traffic volume
AC	Dense-graded asphalt surfaces
ADT	Average Daily Traffic
AGE2	The surface age since construction, expressed in years
ARRB	Australian Road Research Board
AUSTROADS	National Association of Road Transport & Traffic Authorities in Australia
Crackini	Crack Initiation
dTIMS CT	Deighton Total Infrastructure Management System
FWD	Falling Weight Deflectometer
Land Transport NZ	NZTA (from 2004)
LTPP	Long-term Pavement Performance
OGPA	Open-Graded Porous Asphalt
PMS	Pavement Management Systems
RAMM	Road Assessment and Maintenance Management system
SNP	Structural Number of the pavement
Transit	Transit New Zealand
Transfund	Transfund New Zealand (to 2004)
Treatment Length	The sectioning method used to manage New Zealand roads
Von Mises stress	The stress point where yielding occurs

Contents

Executive summary

Introduction

Although the vast majority of road sections in New Zealand consist of thin, flexible chipseal pavements, a large number of vehicle kilometres are travelled on asphalt-surfaced pavements. In addition, the maintenance of asphalt and porous asphalt (OGPA) surfaces takes a significant proportion of the country's overall maintenance funds. This report documents the development of a pavement deterioration model that was undertaken on a sample of dense-graded asphalt- and OGPA-surfaced networks.

This research study is a continuation of previous development work that yielded prediction models that covered mostly thin, flexible chipseal pavements. These chipseal models included:

- crack initiation
- edge-break initiation
- predicting the three stages of rut progression
- maintenance requirements.

The objective of this research report was to extend the model development work to dense-graded asphalt- and OGPA-surfaced pavements. The two types of pavement required different models because the performance, maintenance practices, and decision processes for these surfaces differ significantly.

Given that most European and American countries use significant volumes of asphalt surfaces on their roads, this research includes a full literature review of deterioration models developed for asphalt surfaces elsewhere. Most significant of these are the work that has been completed at the Delft University of Technology under the guidance of Professor A. Molenaar. It was noteworthy to find that some of the latest developments included risk models to predict the probability for ravelling. However, these models are more applicable for design and theoretical applications than for pavement management purposes, as they use data inputs with a high level of detail (Mo et al. 2008).

Other work – also from Europe – included the prediction of asphalt surfaces deterioration using neural networks (Miradi 2004). It is recognised that this technique often results in accurate prediction of performance outcomes, but the limitation is that there is no model expression that can be extracted from the process. This has a practical implication for adopting the outcome in standard pavement management applications. The work was useful in the context of this research, as it suggested significant factors that that could be included in the development of a ravelling model for OGPA surfaces.

Summary of the results

The data used for this research mostly had to rely on network data, as there are few asphalt sections on the LTPP programme. It is thus recognised that models resulting from this study still require validation based on the LTPP programme.

This research successfully achieved the following:

- development of an empirical-probabilistic model that predicts the likelihood of cracking for a dense-graded asphalt pavement
- the use of this model to forecast the probability of an OGPA pavement to ravel
- further work on existing rutting models (Henning et al. 2007) that has confirmed their applicability to in-service asphalt-surfaced pavements – no change to them is required.

Although this research had some excellent outcomes, it is realised that the model development is subject to constant refinement. In addition, it is also realised that implementing the new models into the New Zealand dTIMS system will highlight further research needs.

Further work

Based on the above, and some practical findings of this research, further required work is summarised in the table below.

Table Summary of further research work

Research area	Description of further work
Model refinement	Once more appropriate data is available, the models needs to be refined, including the: • rutting model for asphalt and chipseal pavements • crack-initiation model for OGPA surfaces.
Practical aspects	This research, along with earlier Land Transport NZ research, has highlighted a number of performance and costs issues related to the overlay practices used on OGPA pavements. Best-practice guidelines should be developed that will ensure life-cycle cost efficiencies in these practices.
Including variable section lengths	This research only considered fixed inspection lengths according to a 10% sample. More research is required to investigate the influence of section length on the defect probabilities.
Model adoption	The adoption of these new models into the dTIMS system will also require changes to the decision logic for the system. These changes need to be documented and published with other dTIMS publications.
Further asphalt data needed	Most of the developments in this research had to rely on network data because the LTPP data on asphalt pavements were too limited. The expansion of this programme to include more asphalt pavements is essential.

Abstract

This report details findings from the New Zealand Long-Term Pavement Performance Programme that aims at the development of deterioration models. Earlier work, completed between 2005 and 2007, resulted in prediction models mostly for thin, flexible chipsealed pavements. This report documents the development of models for dense-graded and porous asphalt surfaces (OGPA).

The research was successful in developing pavement deterioration models for crack initiation of dense-graded asphalt surfaces, and ravelling initiation for OGPA surfaces, and has confirmed the validity of the rutting model that was developed earlier. For both the crack and ravelling initiation models, continuous probabilistic models were developed that predict the probability of the defect to occur. These models use data that is readily available on network level databases, and can therefore be applied on asset management applications such as the New Zealand dTIMS system.

The models were tested on the network data and had a significant success rate (up to 75 percent) in predicting the behaviour of the surfaces. Based on this finding, it is recommended that the models are adopted within the New Zealand dTIMS system. Further work required includes refining the models on the basis of individual sections' LTPP data. In addition, this research has highlighted a number of practical aspects that require further investigation, and the need for the development of maintenance best-practice guidelines.

1 Introduction

1.1 Background

Pavement deterioration models form an integral part of pavement management systems (PMS). They provide a PMS with the predictive capability to forecast future maintenance needs and consequential road conditions. As a result, there has been a strong focus and investment into New Zealand pavement model development, in order to satisfy asset management requirements specified by the New Zealand Local Government Act 2002.

During 1998, New Zealand embarked on a national PMS (pavement management system) that combined the off-the-shelf software application, dTIMS (dTIMS CT – Deighton Total Infrastructure Management System), and New Zealand maintenance practices. The dTIMS system consists of a sophisticated optimisation routine that includes predictive capabilities to forecast long-term maintenance needs. In order to achieve this, the system adopted the World Bank HDM-III (and later the HDM-4) pavement condition models. From the onset of the project, the need to calibrate the models to local conditions was realised, and this instigated the establishment of the following New Zealand Long-term Pavement Performance (LTPP) programmes (Henning et al. 2004):

- Transit NZ established 63 LTPP sections on the state highways during 2001, and their condition is surveyed annually – the seventh survey was in April 2008.
- During 2003, Land Transport New Zealand, in association with 21 local authorities, established 82 LTPP sections on typical local authority roads, on both urban and rural networks. Data has been collected on these sections for 5 years.

Since 2006, researchers have been developing new pavement models for local conditions. The initial models were focused mainly on chipseal pavements, since a large portion of the New Zealand network consists of thin, flexible pavement sealed with chipseals. With most of these models completed, the next priority was to develop models that are appropriate for asphalt-surfaced pavements.

This report documents findings from the research on asphalt pavements.

1.2 The status of the development of a New Zealand pavement model

Traditionally, pavement deterioration models predict either an absolute condition value for a given pavement age, or the incremental change of the condition from one year to another. The World Bank HDM-4 model adopted in the New Zealand dTIMS CT system uses both of these approaches. For example, it predicts the expected age at which a pavement will crack as a function of the traffic and other pavement characteristics. After crack initiation, it predicts the incremental increase in cracking from one year to another.

Although these traditional model types provided reasonable answers, New Zealand observations highlighted some differences in the local context. The HDM-4 type models will always predict some degree of pavement deterioration over time, whereas in reality that might not always be the case.

Observations on lower-volume roads suggested that the pavement condition hardly changed over time, but when it did, it deteriorated rapidly. In 2006, Henning et al. proposed a phased approach, as illustrated in Figure 1.1. Later developments in New Zealand focused more on the timing of certain significant stages in pavement life, rather than the change in condition from one year to another. This timing is not predicted as an absolute outcome, but rather as a probability.

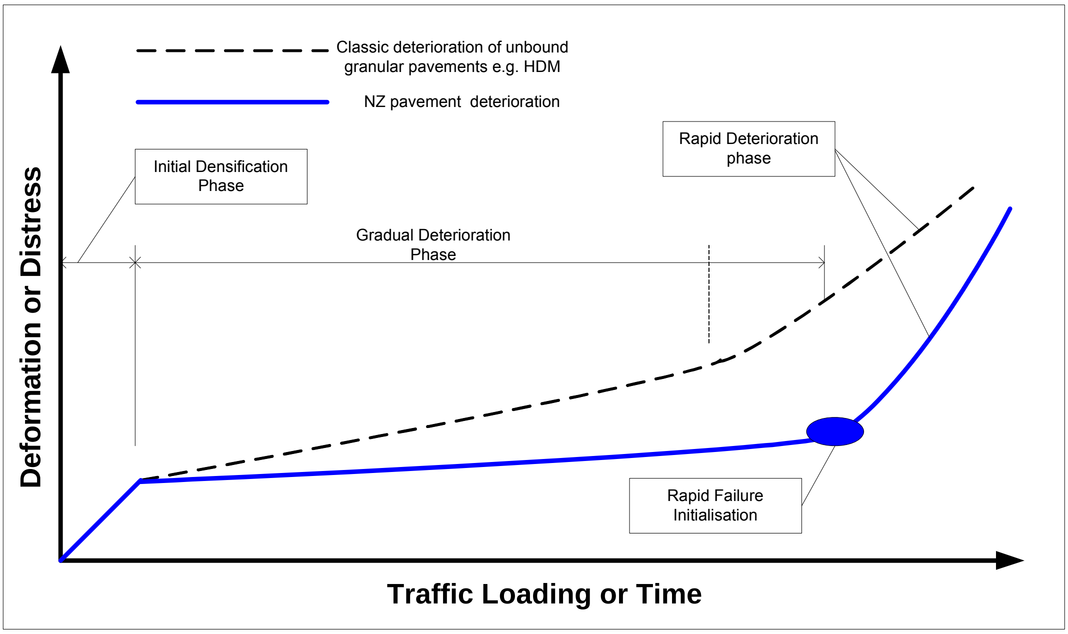


Figure 1.1 Comparing the default HDM rut model with observed rut progression in New Zealand

As seen from this figure, rutting is predicted according to three phases:

1. initial densification
2. stable or constant rut change
3. initiation of accelerated rut progression.

In order to predict the initiation of defects, Henning et al. (2006) proposed the use of the logistic model. This model format predicts the probability of a defect to begin, rather than the absolute timing. As a result, this model format promises two major advantages:

- It uses the time/use factor as an independent variable. When some variables, such as bitumen hardening or aging, are not available to the model, the time/use factor acts as a surrogate of the unknown factors.
- Because it predicts the probability of failure throughout the life of the pavement, it provides significant flexibility to the modelling process. For example, it is possible to predict different network level risk profiles as a result of different maintenance investment strategies.

This probabilistic model format is already widely accepted and used in New Zealand, and is used to predict:

- crack initiation

- edge-break initiation
- accelerated rut progression
- maintenance requirements.

1.3 Objectives of this research

The purpose of this research was to develop the main pavement deterioration model for asphalt pavements in New Zealand. The models would differentiate between dense-graded asphalt surfaces (AC) and porous asphalt surfaces (OGPA). The types of models considered for this research included the main drivers for maintenance decisions and included:

- cracking
- rutting
- ravelling.

Skid resistance was excluded from this research, as there were a number of other studies in this area already in progress.

1.4 Data use

As this was the first research into the performance of asphalt pavements in New Zealand, the focus was on developing models of the main maintenance drivers for asphalt pavements, as mentioned above. The priority models that were to be developed are summarised in the table below. Table 1.1 lists the work required on the individual defects, plus assumed data sources that were utilised for the respective analysis.

Table 1.1 Model development work summary and data sources

Pavement model/ maintenance driver	Description of work required	Data considerations
Ravelling	• Consider probability model for initiation • Test progression	• Network data • Test on LTPP
Cracking	• Review exploratory statistics for AC and OGPA to identify main variables • Regression analysis	• Transit Auckland and Wellington network data
Rutting	• Confirm rutting expressions based on revised SNP principle	• Detail per LTPP sections

Note: This research mostly relied on a combination of data sourced from the LTPP programmes and from network data. There are a combined total of 15 LTPP sections that consist of asphalt-surfaced pavements. It is therefore realised that the data may not be sufficient for all the model developments.

2 Literature review

This section contains a brief overview of the literature review that investigated pavement models developed for asphalt pavements. Our aim was to investigate existing model formats and to note significant variables commonly used in other models. The subsequent sections discuss some of the literature for the respective models.

2.1 Cracking

2.1.1 Cracking mechanisms

There are a number of failure mechanisms associated with the occurrence of cracking in asphalt pavements. Some of the most common mechanisms are summarised in table 2.1.

Table 2.1 Crack mechanisms in asphalt pavements

Crack type	Mechanism	Reference
Transverse thermal or Visco-elastic fracture	This type of cracking is normally associated with the combined effect of asphalt stiffness and external strain factors, such as significant temperature changes or movements within the underlying layers. This failure mechanism is therefore a result of tensile stresses exceeding the tensile strength of the asphalt. These cracks are easily identified, since they are normally long, straight cracks that are orientated in a transverse direction across the road.	Molenaar 2004
Load-associated or fatigue cracking	According to most mechanistic design methods, failure of an asphalt layer is defined as the time when crack initiation occurs. At this point, the permissible tensile strain at the bottom of the asphalt layer is exceeded because of repetitive loading. Factors that affect the strain conditions at the bottom of the asphalt layer include: • stiffness of the asphalt • thickness of the asphalt layer • support from underlying layers, including the sub-grade. Load-associated cracks can manifest in a number of ways, but would normally be located within the wheelpath. In addition, those cracks would not be straight, and are often referred to as 'alligator cracks'.	AUSTROADS 2004
Reflective cracking	Reflective cracking is due to stress concentrations that exist because of underlying layers that are already cracked. This mechanism is frequently found in overlaid layers on top of existing concrete layers or cracked pavements. Reflective cracks would normally have the same crack pattern as the underlying cracks.	NDLI 1995

This report mostly deals with load-associated cracks, which normally require more expensive maintenance types such as overlays or full-depth rehabilitations. The other crack types would normally be sealed using a crack-sealing agent.

2.1.2 Modelling approaches

Three main crack-initiation prediction models were found during the literature review, and can be categorised according to:

- deterministic models based on performance data
- mechanistic design models
- probabilistic-mechanistic models.

These modelling approaches are further discussed in the subsequent sections.

2.1.2.1 Deterministic models based on performance data

The most widely used models of this type are the HDM-III and HDM-4 crack-initiation models. According to the HDM definition, crack initiation occurs when a surface displays cracks on more than 0.5% of its area (Watanatada et al. 1987). The cracked area is calculated by multiplying the length of the crack by the width of the affected area (for line cracks, the affected width is assumed to be 0.5 m). There are separate crack-initiation model forms in HDM-4 for stabilised and granular bases, and for original surfaces and resurfaced surfaces. In contrast to global practice, the majority of New Zealand roads fall into the granular-base category, as most New Zealand pavements are only lightly stabilised. The crack initiation for these types of pavements can be predicted as follows (from NDLI 1995):

Original surfaces:

$$ICA = K_{cia}\left\{CDS^2 a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\} \quad \text{(Equation 2.1)}$$

Resurfaced surfaces:

$$ICA = K_{cia}\left\{CDS^2\left[MAX\begin{pmatrix} a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] * \\ MAX\left(1 - \frac{PCRW}{a_3}, o\right) a_4 HSNEW \end{pmatrix}\right] + CRT\right\} \quad \text{(Equation 2.2)}$$

where:

ICA	time to initiation of ALL structural cracks (years)
CDS	construction defects indicator for bituminous surfaces
YE4	annual number of equivalent standard axles (millions/lane)
SNP	average annual adjusted structural number of the pavement
HSNEW	thickness of the most recent surface (mm)
PCRW	area of all cracking before latest reseal or overlay (% of total cracking area)
K_{cia}	calibration factor for initiation of all structural cracking

CRT crack retardation time because of maintenance (years)

a_i model coefficients (may vary for pavement types)

The expressions basically consist of a structural crack component that is dependent on the SNP and $YE4/SNP^2$. This value is multiplied by the percentage of previous cracking, and for resurfaced sections, the thickness of a new surface.

2.1.2.2 Mechanistic design models

The mechanistic design methods usually use a calibrated transfer function that determines the number of load repetitions possible for a given asphalt layer. For example, AUSTROADS (2004) gives the number of possible load repetitions on an asphalt layer as:

$$N_{lab} = \left| \frac{6918(0.856 V_B + 1.08)}{(S_{mix}{}^{0.36} \mu\varepsilon)} \right|^5 \qquad \text{(Equation 2.3)}$$

and

$$N_{field} = \text{"}ShiftFactor\text{"}\, xN$$

where:

Smix Stiffness Modulus of Mix (MPa)

VB volume of bitumen (%)

με permissible (micro) strain for a given number of repetitions, N

It is accepted that the mechanistic design method provides for theoretical design applications. For that reason, it is also accepted that it consists of variables that are not normally available in a performance prediction application. However, it is worth noting the variables that influence the performance.

2.1.2.3 Probabilistic-mechanistic models

Bouwmeester et al. (2004) introduced one of the first probabilistic performance models for asphalt pavements. This method differs significantly from the New Zealand crack-risk models introduced by Henning et al. (2006). The New Zealand development uses the logit model format to predict the probability of cracking in chipseal format, using the following expression:

$$p(stat.aca) = 1 \Big/ \left[1 + \exp\begin{pmatrix} -0.141AGE2 + \{(5.062, 3.440)\text{ for } stat.pca = (0,1)\} \\ -0.455Log(AADT) - 0.275Log(HTOT) + 0.655SNP \end{pmatrix} \right]$$

(Equation 2.4)

where:

p(stat.aca) the probability of a section being cracked

AGE2 surface age in years, since construction

stat.pca cracked status prior to resurfacing (0 for uncracked, 1 for cracked)

HTOT	total surface thickness (in mm) of all the layers
AADT	annual number of equivalent standard axles (millions/lane)
SNP	modified structural number

Bouwmeester et al. (2004) used the traditional risk approach of likelihood and consequences, combined with a stochastic model to define the risk of cracking in asphalt pavements. The advantage of this approach is that it allows for the variability in loading and in the existing strength of the asphalt pavement.

The final expression is given by Bouwmeester et al. (2004):

$$\begin{aligned} Z &= R - S \\ &= \log k_1 - n\log\varepsilon + \log(H0 + \log(V) - (I \cdot W \cdot F_s \cdot F_{nd} \cdot D_{traffic} \cdot G) \end{aligned} \qquad \text{(Equation 2.5)}$$

where:

Z	reliability [SAL_{100}]
R	practical strength [SAL_{100}]
S	design load [SAL_{100}]
$\log k_1$	material characteristic
n	material characteristic
ε	strain at the bottom of the asphalt layer
H	healing factor
V	factor related to traffic wander
[SAL_{100}]	100 kN dual wheel axle

According to this approach, the failure/cracking probability of the asphalt is graphically represented in figure 2.1 on the next page, while the probability density of both the design load and the strength is depicted in figure 2.2.

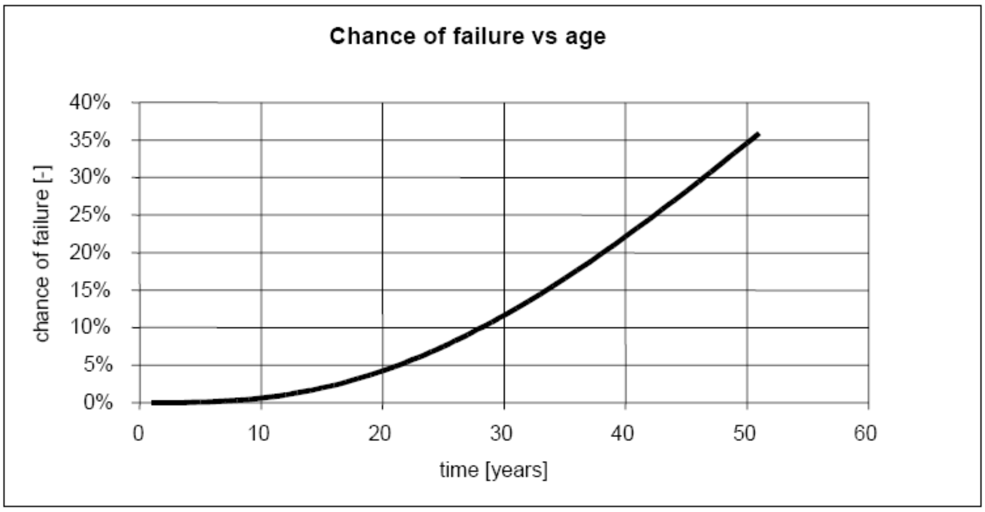


Figure 2.1 **Increased failure probability with age of asphalt pavements (Bouwmeester et al. 2004)**

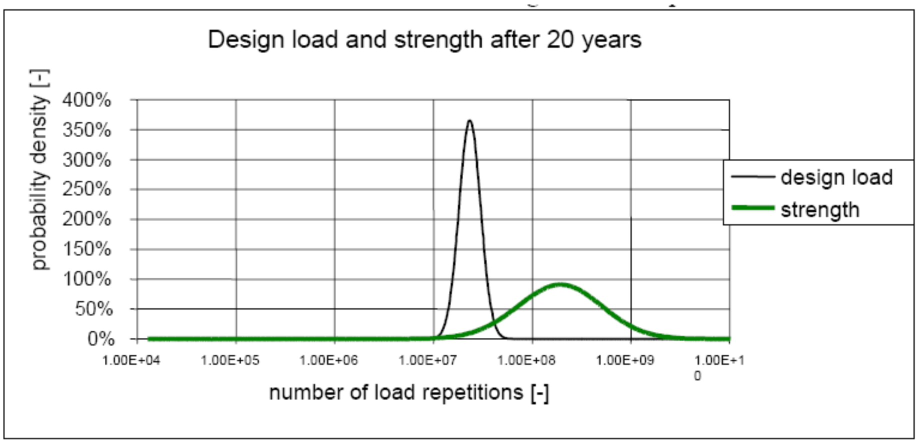


Figure 2.2 **Design load and strengths after two years (Bouwmeester et al. 2004)**

It is noted from the two figures that a design period of 20 years was assumed. At 20 years, the probability of failure is only at 5 percent, which seems low. It is also noted that the probability density for the design load is not as widely distributed as the strength of the asphalt layer.

2.2 Ravelling of porous asphalt surfaces

Porous asphalt – known as ‘Popcorn asphalt’, ‘Whisper asphalt’ or ‘Open-graded porous asphalts’ (OGPA) – is a popular surface on motorways, especially through urban areas. Compared to other asphalt mixes, OGPA provides two main benefits:

1. a smooth surface that absorbs most of the sound caused by the tyre/road surface contact
2. a free-draining surface that allows the water to penetrate the top surface and then to be removed on the underlying dense-graded asphalt surface – this provides good skid resistance for vehicles, and prevents water spray behind moving vehicles.

Figure 2.3 illustrates the particle size make-up of OGPAs.

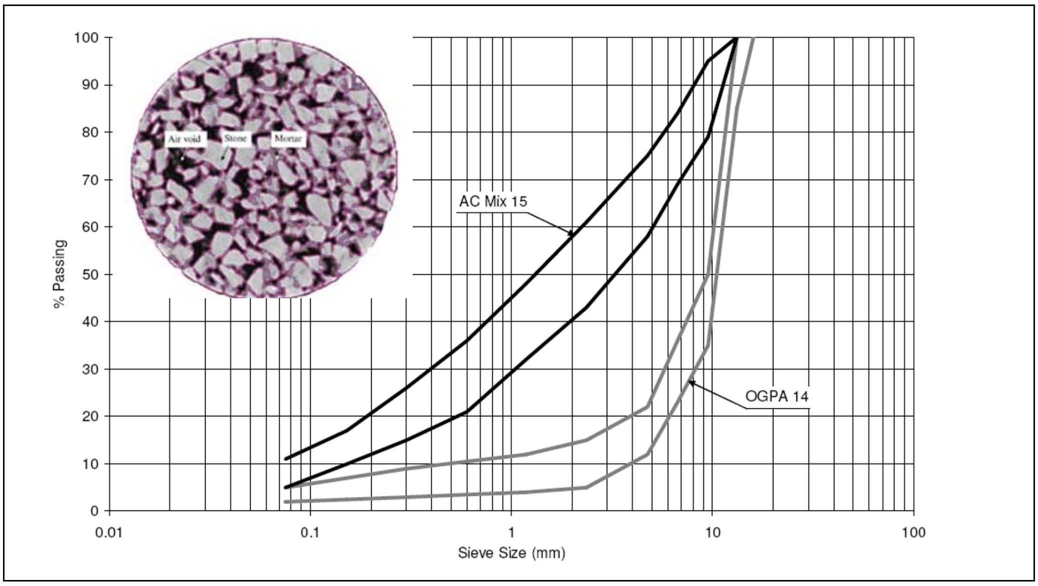


Figure 2.3 Particle size make-up of a porous asphalt mix (Theron and Fletcher 2007; Mo et al. 2008)

The figures indicate that the main characteristic of an OGPA surface is single-graded stones, without the filler material normally found in other asphalt mixes. This grading results in a high percentage of voids in the mix. In order for the OGPA to be functional, it relies on the stone-on-stone contact for compression strength, and on the bitumen tensile strength for adhesion between the particles. The stiffness of the OGPA layer would therefore be a function of these factors combined with the stiffness of the underlying layers.

The main failure mechanisms for this surface are either cracking or ravelling of the top layer. During the survival analysis of the Auckland network, Henning et al. (2008) estimated that 70 percent of surface replacements/overlay are triggered by OGPA ravelling. The following sections discuss ravelling modes and modelling completed in international studies.

2.2.1 Ravelling mechanism

The ravelling of OGPA is primarily caused by the failure of the inter-particle bonding provided by the bitumen film. This may result from normal fatigue of the layer due to the working of repetitive vehicle loadings, or due to premature failure as a result of poor design or construction processes. For example, if the stiffness of the bitumen is too high, it is expected to be brittle and more prone to breaking. By investigating the bitumen bonding between particles, figure 2.4 illustrates a meso-scale model of porous asphalt, in an attempt to model ravelling.

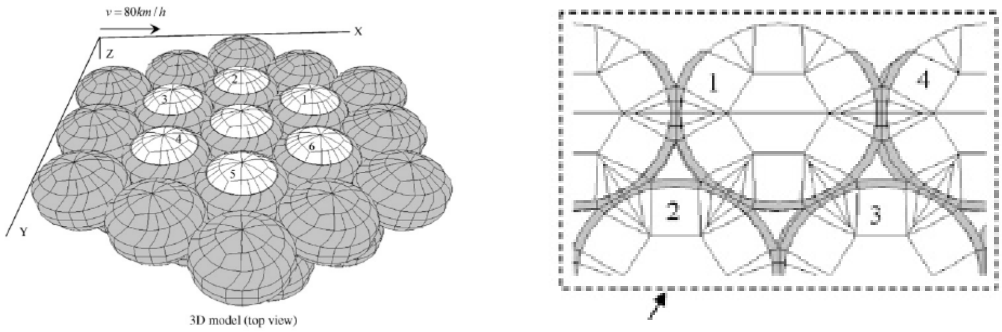


Figure 2.4 Meso-scale models constructed to simulate OGPA layers (Mo et al. 2008)

It can be observed from the figure that the contact area between particles is relatively small; thus the allowable tensile stresses are relatively low. The outcome of the modelling is discussed in the next section.

2.2.2 Modelling approaches

2.2.2.1 The World Bank – HDM ravelling model

The World Bank HDM ravelling model is given by NDLI (1995):

$$IRV = K_{vi}CDS^2a_0RRF\exp(a_1YAX) \qquad \text{(Equation 2.6)}$$

where:

IRV	time to ravelling initiation
K_{vi}	calibration coefficient for ravelling initiation
CDS	construction defect indicator for bitumen surfacing
RRF	ravelling retardation factor because of maintenance
YAX	annual number of axles of all motorised vehicle types in the analysis year (millions/lane)
a_0&a_1	model coefficients

In essence, the HDM ravelling model is simply a function of the traffic carried by the surface. The bitumen condition and/or age do not become part of the model, other than allowing maintenance such as rejuvenation treatments, which will defer ravelling initiation.

2.2.2.2 Findings for the University of Delft

Mo et al. (2008) have developed rather complex models to predict ravelling of OGPA surfaces by analysing the stresses of the inter-particle bitumen film (see figure 2.4). A finite element analysis model was developed to determine the stress and strains resulting from loading on the porous asphalt layer model. As a result of this process, the Von Mises stress in the material was developed for the various loading times, as illustrated in figure 2.5.

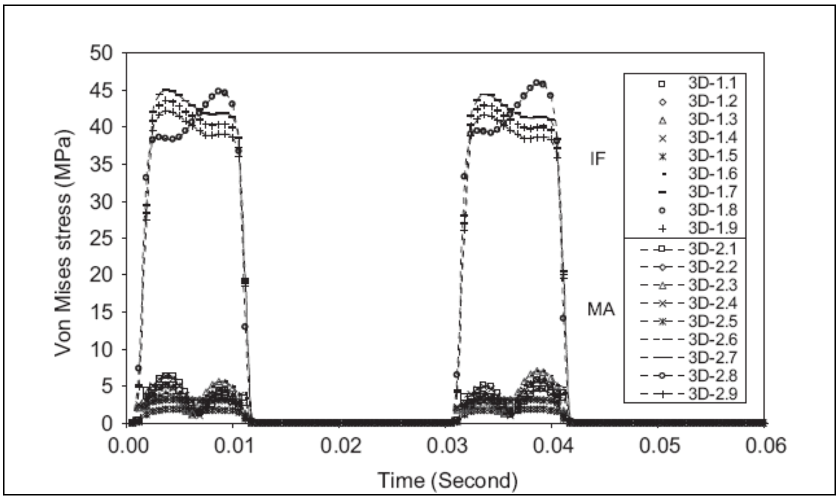


Figure 2.5 Von Mises stress in mortar for each contact region over time in 3D model (Mo et al. 2008)

In terms of benefit to this study, the results from the Mo study indicated that the critical factors leading to ravelling include the:

- loading conditions – e.g. the propelling wheel would induce the critical load conditions
- visco-elastic behaviour of the mix, which naturally affects the inter-particle strains
- particle packing – the most significant factor that influences the stress and strain conditions of the mortar.

2.2.2.3 Modelling ravelling using neural networks

In her 2004 paper, Miradi explains a process for modelling porous asphalt using neural networks. A neural network is a self-learning system that considers all factors in predicting an outcome of a variable. These predictions are based on historical outcomes that were caused by a certain combination of factors. With the introduction of more data, the system learns more about the decision process, and the main drivers causing the various outcomes.

The main benefit of this system is that it can accurately load relative weightings/importance to each factor that may influence an outcome of the system. However, the neural network process does not provide a prediction model that can be used in an external system. It therefore remains unclear what relation each factor has to the independent variable.

Figure 2.6 illustrates the outcome from the neural network analysis completed for ravelling on porous asphalt surfaces.

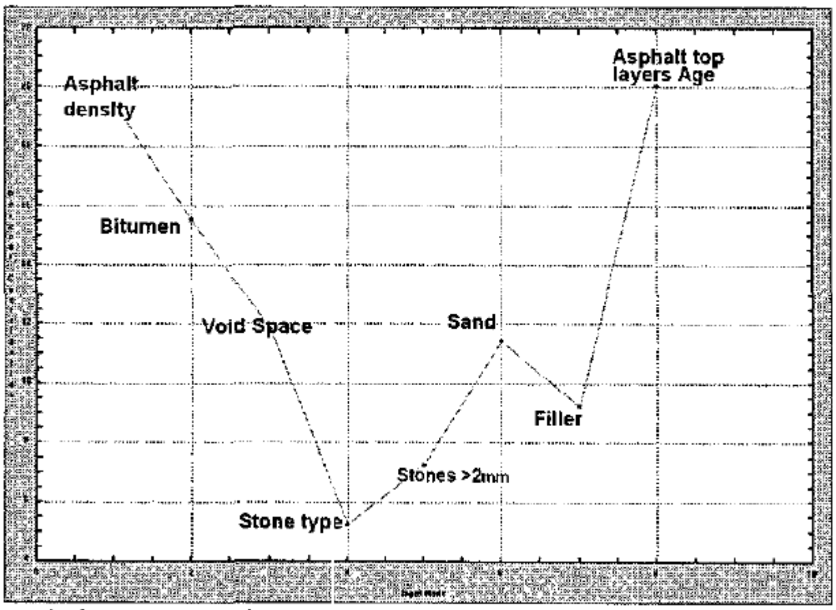


Figure 2.6 Output obtained from neural networking completed on porous asphalt ravelling (Miradi 2004)

It is observed from this figure that the most significant factors influencing ravelling are:

- surface age
- asphalt density
- bitumen
- void space.

As with other studies on porous asphalt, most factors influencing ravelling are related to the asphalt make-up. It seems that these studies do not include any long-term performance factors such as traffic, or the pavement underlying these layers.

2.3 Rutting

2.3.1 Previous work completed

Section 1.2 explained the shift in the approach of predicting rutting in New Zealand. This modelling development mostly took place on the basis of the CAPTIF data that was calibrated to the LTPP sections.

The models that were developed for predicting rutting include (Henning et al. 2007):

2.3.1.1 Predicting the initial densification rut

$$\text{Initial Rut} = 3.5 + e^{(2.44 - 0.55\,\text{SNP})} \qquad \text{(Equation 2.7)}$$

where: SNP is the structural number as derived from the Falling Weight Data.

2.3.1.2 Predicting the rut progression

Thin pavements:

$$RPR = 9.94 - 1.38 \times a_1 SNP \qquad \text{(Equation 2.8)}$$

Thick pavements (>150mm):

$$RPR = 14.2 - 3.86 \times a_2 SNP \qquad \text{(Equation 2.9)}$$

where:

RPR stable rut progression rate in mm/million ESA

SNP modified structural number

a_1, a_2 calibration coefficients

2.3.1.3 Predicting the initiation of accelerated rutting

$$p(\text{Rutaccel}) = \frac{1}{1 + e^{\left(-7.568\times10^{-6}\times ESA + 2.434\times snp - \left[(4.426, 0.4744)\text{ for thickness} = (0,1)\right]\right)}}$$

(Equation 2.10)

where:

ESA equivalent standard axles

SNP pavement strength structural number

Thickness 0 for base layer thickness <150 mm
1 for base layer thickness >150 mm.

It was not expected that our research would result in a significantly different model to the above, but analysis had to investigate a potential refinement of the model for in-service asphalt pavements.

3 Dataset for this research

3.1 Description

The dataset for this research consists of a total of 4968 sections, with roughly 50 percent of these being split between dense-graded asphalt and porous asphalt surfaces (OGPA). The data has been sourced from Transit's state highway networks for Auckland and Wellington. Brief descriptions of the data characteristics are presented in the following sections.

3.1.1 Rating data

Visual distress data, such as cracking and ravelling, were sourced from the RAMM rating surveys that are undertaken on 10 percent of each treatment length across the entire network (HTC 1999). Treatment lengths on the state highways typically vary between approximately 200 m to 2 km. The rating sections are taken as the first 50 m of each 500-metre length. Visual distress data are expressed as a function of the wheelpath length affected. No classification is made regarding the severity of distresses. Therefore, once a crack appears, it is rated regardless of the width of the crack.

For this research, all the visual data was converted into a percentage scale consistent with the approach followed for HDM-III and HDM-4. Defect initiation was also defined as the occurrence of a defect at the point it reaches 0.05 percent.

The outcomes of this research are constrained according to the limitations of the current RAMM rating process, which is under revision. The main limitation in terms of this research is the RAMM survey sampling. One would expect different initiation probabilities for varying section lengths – it is unclear whether the 10 percent sampling approach of the RAMM rating surveys will account for this effect.

The outcome of initiation models would be affected by the length of the sections. It would therefore be necessary to review the results of this research when more data becomes available.

3.1.2 High-speed data

The high-speed data on the state highways report data at a 20-metre interval and are summarised to the treatment lengths (TL). Although the high-speed data provides accurate readings, previous work has suggested that the repeatability is not always adequate for modelling development work (Henning et al. 2004). This limitation would be of concern for the rutting model review. However, the current model would be unchanged if the variation in the results are too great.

3.1.3 Strength data

This project has utilised the strength data based on Falling Weight Deflectometer (FWD) surveys on the state highway network. The FWD data has been analysed to provide a modified structural number (SNC) that was computed according to a New Zealand regression method (Salt's method) published in HTC (2000). In addition to this, a structural index, SIrut, was trialled as part of another project, and was used as an alternative strength parameter (Salt et al. 2008). The SIrut has been developed according to deflection measurements and strain failure criteria according to the AUSTROADS (2004) design method.

All available strength and deflection parameters, such as pavement layer indices, were used during the regression analysis. In each instance, the parameter that correlated the best with the dependent variable was used for the final models.

3.2 Descriptive statistics

The descriptive statistics give an overview of the dataset. The main reason for investigating the descriptive statistics is to understand the range and distribution of the dataset. A brief summary of the main data items is presented below. Figure 3.1 illustrates the distribution of the average annual daily traffic (ADT) in both directions and the structural number (SNP).

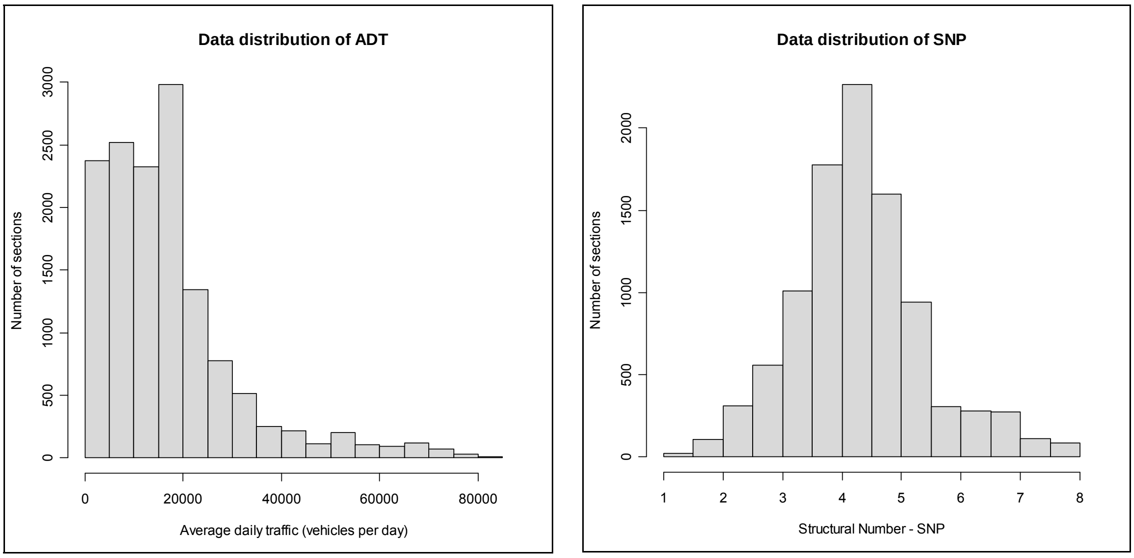


Figure 3.1 Distribution of the traffic and pavement strength (SNP)

As expected, the motorway sections of both Auckland and Wellington represent the most highly trafficked roads in New Zealand. The majority of sections carry up to 20,000 vehicles per day. Consequently, it is also expected that these pavements would have a relatively high structural make-up. The graph indicates an average SNP that is higher than 4. There is a concern about the pavements with structural numbers of less than 3, which would not be adequate for the indicated traffic volumes.

Figure 3.2 illustrates the make-up of surfaces on these two networks. It is evident that a large portion of the network consists of asphaltic surfaces that are deeper than 50 mm. These sections mostly consist of surfaces with dense-graded asphalt combined with porous asphalt, multiple OGPA layers, and structural asphalt layers with a thickness of more than 80 mm.

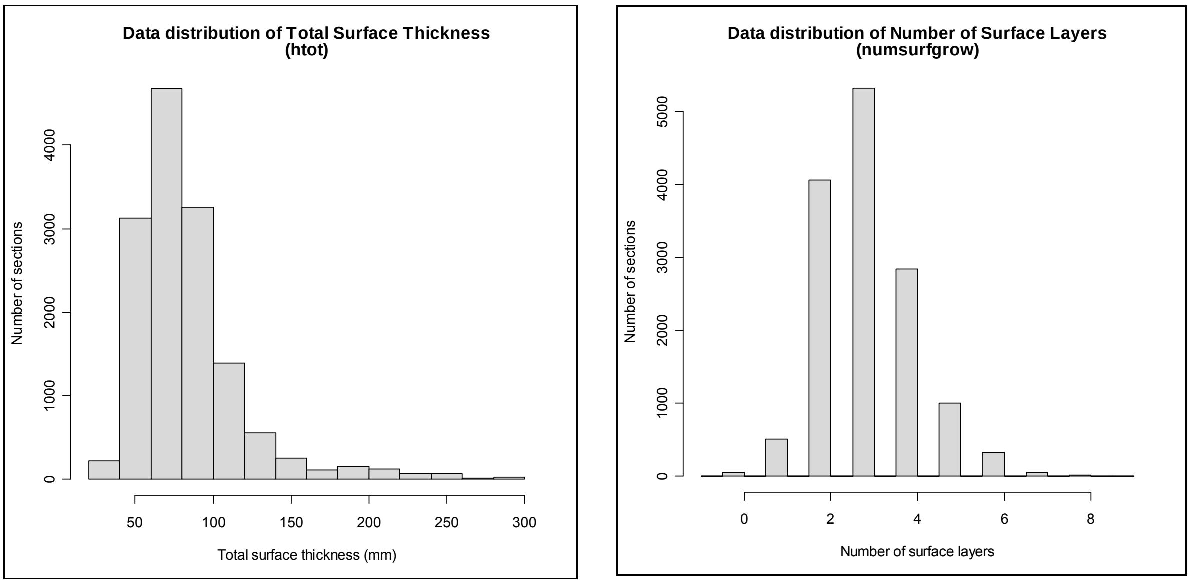


Figure 3.2 Distribution of the total surface thickness and number of surfaces

The figure also depicts the number of surface layers per section. The majority of sections have more than two surface layers. It is also evident that there are a number of sections with more than four surface layers.

Figure 3.3 illustrates the current average age of the surfaces on the network. There is an even distribution of the surface ages, with a maximum of 14 years.

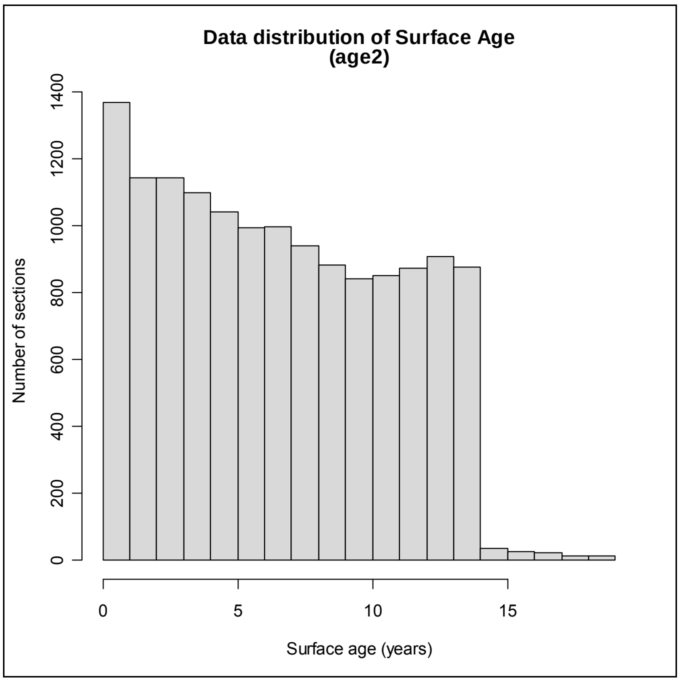


Figure 3.3 Distribution of the surface age

4 Crack initiation

4.1 Objectives of the development of a crack-initiation model

Based on the experience of the New Zealand crack-initiation models for chipseal pavements, plus the latest international trends, the most logical cracking model for asphaltic surfaces would be of a probabilistic format. It would be practical if this model had a format consistent with the chipseal pavement model.

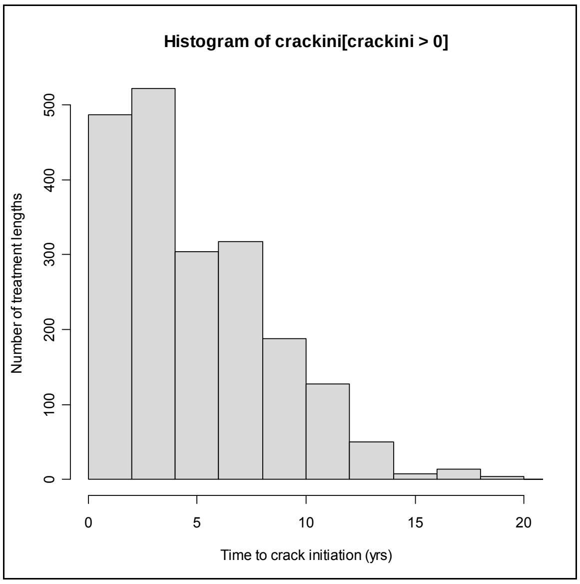
However, it is realised that the asphalt crack mechanism would differ significantly from that of the chipseal pavements. The literature review has confirmed that asphalt cracking is driven from layer stiffness versus traffic loading perspectives. The objectives of the development of a crack-initiation model would therefore be to:

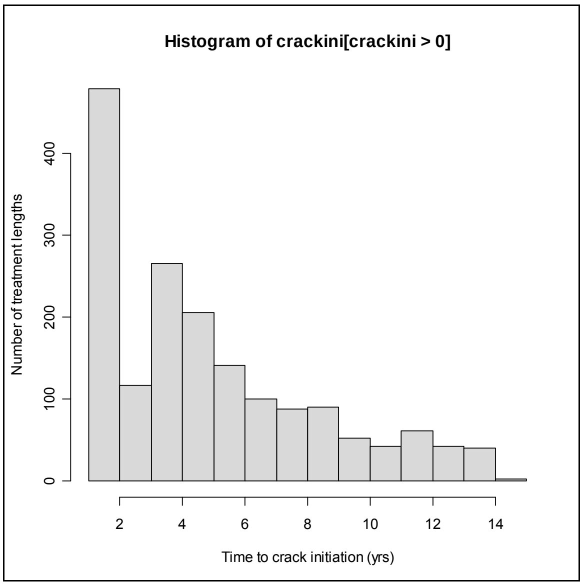
- establish the significant factors influencing the crack initiation of asphalt- and OGPA-surfaced pavements
- use these significant factors, determined in a regression analysis, to develop the crack probabilistic model.

The results from the analysis are presented in the following sections.

4.2 Exploratory statistics

Figure 4.1 presents the distribution of crack initiation for dense-graded asphalt and OGPA pavements. In both instances, there is a logarithmic trend for the number of sections that cracked at different surface ages. In both cases, most sections had cracked before an age of 5 years, which is surprisingly early.

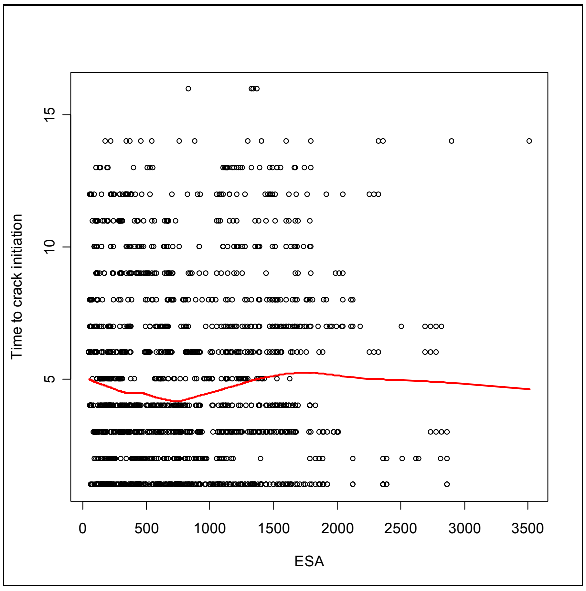


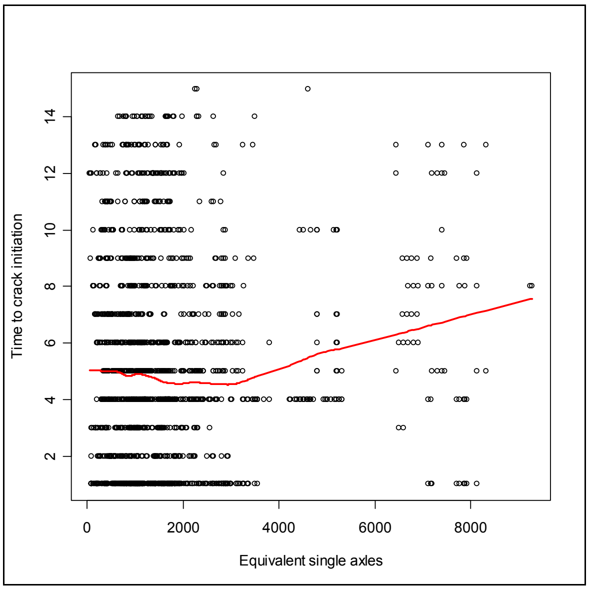


Asphalt **OGPA**

Figure 4.1 Crack initiation as a function of surface age

Figure 4.2 below illustrates the relationship between the crack initiation and traffic loading. It is observed that there is no apparent relationship between loading and crack initiation on asphalt-surfaced pavements. The relationship is somewhat stronger for OGPA surfaces, although higher traffic volumes suggest longer crack initiation times.





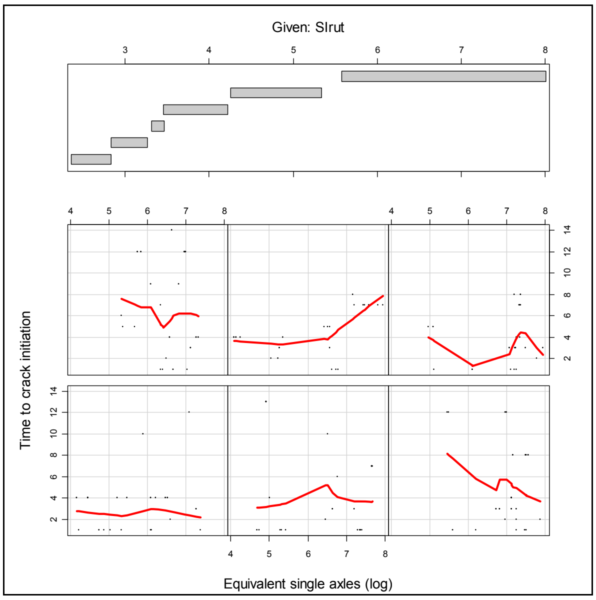
Asphalt **OGPA**

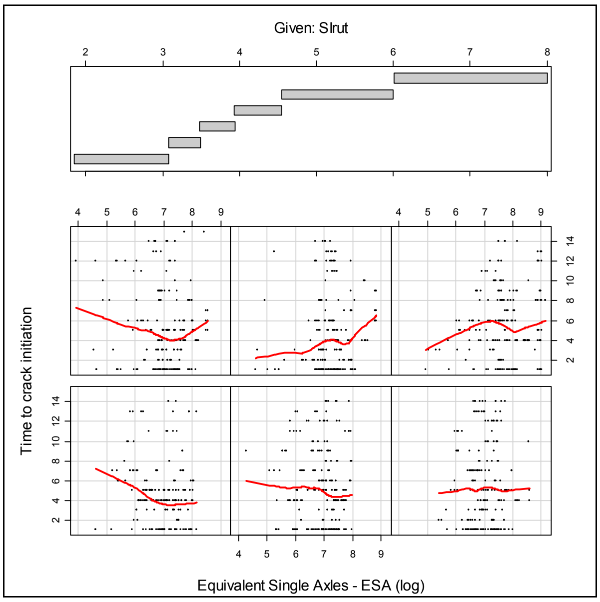
Figure 4.2 Crack initiation as a function of traffic loading (ESA per day)

The observations made in figure 4.2 are counter-intuitive, since it would be expected that higher traffic loading would result in faster crack initiation. One of the potential explanations for this discrepancy is that most of the road pavements from the study area were adequately designed for the traffic loading they were to carry. Therefore, the crack initiation has occurred due to other factors, not related to the traffic loading.

Figure 4.3 illustrates the relationship between crack initiation and traffic loading combined with structural number. The figure shows the relationship between ‘time to crack initiation’ and ‘equivalent single axles’ in the plots, which are divided into six classifications for the ‘rutting strength index’ (SIrut).

Again, this figure illustrated varying trends for different strength pavements, confirming that traffic loading and strength are not significant factors in predicting crack initiation for both AC and OGPA pavements.

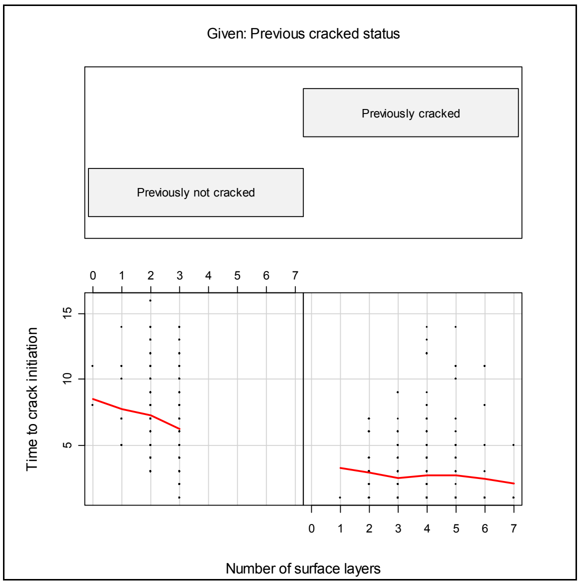




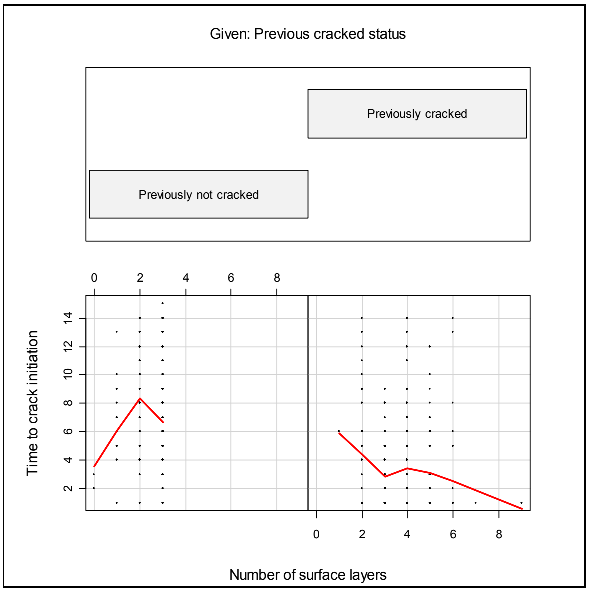
Asphalt **OGPA**

Figure 4.3 Crack initiation as a function of structural rutting index (SIrut) and traffic load (ESA)

Figure 4.4 illustrates two of the more significant predictors of crack initiation for both the asphalt surfaces. In the first instance, it is clear that the cracked status prior to an asphalt overlay significantly influences crack initiation of the subsequent asphalt surfaces. In addition, as the number of overlays increases, the time elapsing before crack initiation decreases. For AC surfaces, the crack status has a stronger relationship compared to the number of surface layers. On the other hand, it seems that OGPA surfaces are more influenced by the number of underlying surfaces than the crack status prior to overlay.



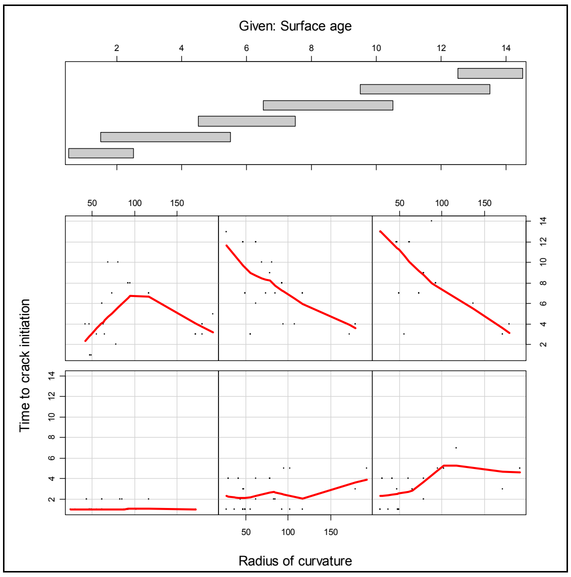
Asphalt



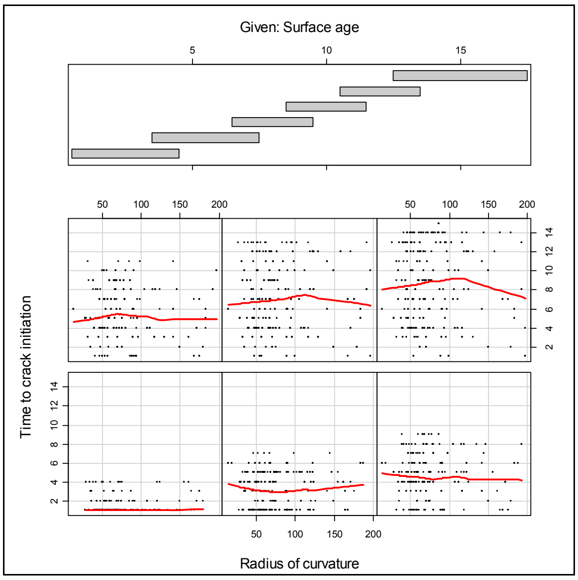
OGPA

Figure 4.4 Crack initiation as a function of crack status prior to resurfacing and number of surface layers

Figure 4.5 illustrates the relationship between surface age and radius curvature with the crack initiation time. Here, there is a clear difference between AC and OGPA surfaces. There is a stronger relationship between surface age and radius of curvature for AC surfaces. With an increase in surface age, crack initiation is faster on more flexible pavements. There is only a slight trend observed on the OGPA surfaces.



Asphalt



OGPA

Figure 4.5 Crack initiation as a function of surface age and radius of curvature

4.3 Predicting crack initiation for asphalt surfaces

4.3.1 Dense-graded asphalt

Table 4.1 presents the result from the regression, while the resulting expression is provided in equation 4.1 following.

Table 4.1 Regression outputs for asphalt crack initiation

	Estimate/ coefficient	**Std. error**	**z value (sample variance)**	**Pr(>|z|) (confidence interval)**	**Significance**
(Intercept)	-2.277	0.435	-5.23	1.70E-07	***
H_{new}	0.008	0.006	1.284	0.19914	
factor(PCA)1	3.900	0.462	8.431	< 2e-16	***
AGE2	0.228	0.0160	14.227	< 2e-16	***
R	0.001	0.000	2.075	0.03801	*
Factor(PCA)0: Surfnum	-0.003	0.1250	-0.024	0.98102	
Factor(PCA)1: Surfnum	-0.678	0.0856	-7.913	2.51E-15	***
Log(ESA):SI_{rut}	-0.020	0.007	-2.836	0.00457	**

Significance codes:

'***' = 0, '**' = 0.001, '*' = 0.01, '.' = 0.05, ' ' = 0.1

$$P_{CIAC} = 1 \Big/ \left[1 + \exp\left(-0.228\,AGE_2 + kci_{ac} \begin{Bmatrix} 2.277 - 0.008H_{new} - 3.9PCA_1 - 0.001R + \\ Surfnum(0.003PCA_0, 0.678PCA_1) + \\ 0.02Log(ESA) * SI_{rut}) \end{Bmatrix} \right) \right]$$

(Equation 4.1)

where:

Hnew	the thickness of the top surface layer
Factor(PCA)	the cracked status of the previous surface layer (0 = false, 1 = true)
AGE2	surface age
Surfnum	number of surface layers
ESA	average equivalent standard axles per day

SI_{rut} structural rutting index

Note that the SI_{rut} is similar to the structural number and is derived from deflection data.

The regression results confirmed the observations from the exploratory statistical analysis. The three most significant factors predicting crack initiation for AC surfaces were surface age, cracked status of the previous surface layer, and cracked status related to the number of surface layers.

Figure 4.6 and Figure 4.7 present the outcomes of the model developed for AC surfaces. They show the probability for a surface to be cracked within a given year for two scenarios: cracked, and not-cracked. The solid line is the probability of cracking for overlaid surfaces when the previous surface was cracked. The broken line represents surfaces for new pavements or overlays on uncracked surfaces. Figure 4.6 suggests that an asphalt overlay on top of an existing cracked surface will not take very long to crack again. On average, new surfaces will take more than 10 years before crack initiation.

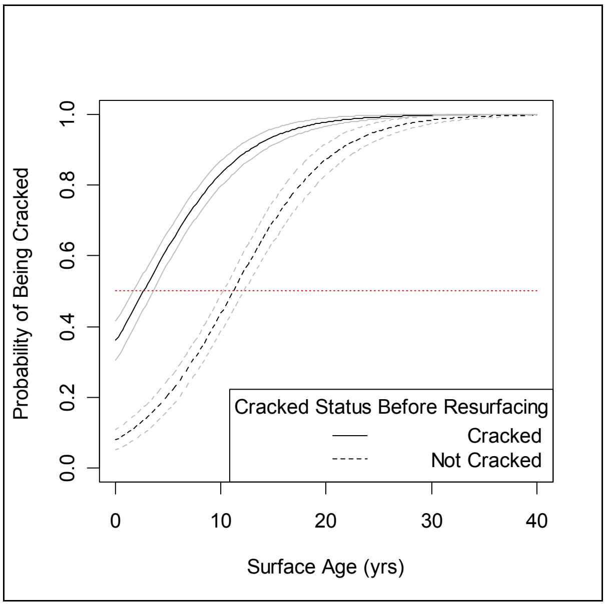


Figure 4.6 Predicting crack initiation for dense-graded asphalt surfaces

Figure 4.7 was developed to try to test the ability of the model to correctly predict crack initiation. This figure plots the number of right and wrong predictions for sections being either cracked or not cracked. In total, the model was capable of correctly predicting the cracked status of pavements for 62 percent of cases. This figure suggests an acceptable result for the AC surfaces.

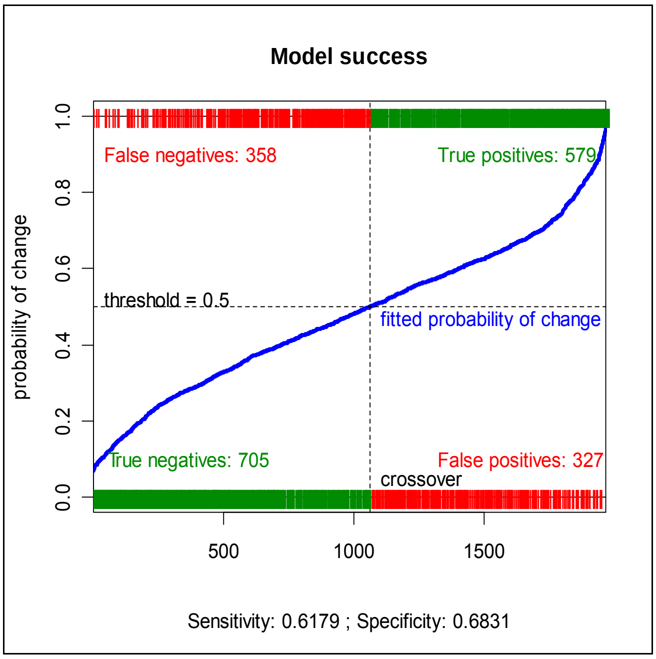


Figure 4.7 Testing the success of the asphalt surface crack-initiation model

Notes:

False negative	predicted to be uncracked, but actually it is cracked
True positive	a section has been correctly forecast as cracked
True negative	both the predicted and actual status are uncracked
False positive	the section has been predicted to crack, but it is uncracked.

4.3.2 Porous asphalt (OGPA)

The regression on the OGPA surfaces resulted in a less satisfactory modelling outcome. A number of significant factors were identified, as presented in table 4.2. However, both figure 4.8 and figure 4.9 indicated less than ideal results. Figure 4.8 shows crack initiation times that are well outside the expected ranges when compared with dense-graded AC.

Table 4.2 Regression outputs for OGPA crack initiation

	Estimate/ coefficient	Std. error	z value (sample variance)	Pr(>|z|) (confidence interval)	Significance
(Intercept)	-0.63434	0.21403	-2.964	0.00304	**
AGE2	0.174152	0.009115	19.106	< 2e-16	***
SI_{rut}	-0.60557	0.089023	-6.802	1.03E-11	***
R	0.000125	0.000314	0.399	0.68994	
SI_{rut}:log(ESA)	0.05374	0.00992	5.418	6.04E-08	***
factor(PCA)0: Surfnum	-0.56387	0.060252	-9.359	< 2e-16	***

factor(PCA)1: Surfnum	-0.0887	0.034855	-2.545	0.01093	*

Significance codes:

'***' = 0, '**' = 0.001, '*' = 0.01, '.' = 0.05, ' ' = 0.1

$$P_{CIAC} = 1 \Big/ \left[1 + \exp\left(-0.174\,AGE_2 + Kci_{OGPA} \left\{ \begin{array}{l} 0.634 - 0.0001R + 0.606\,SI_{rut} \\ + Surfnum(0.564\,PCA_0, 0.089\,PCA_1) - \\ 0.054\,Log(ESA) * SNP) \end{array} \right\} \right) \right]$$

(Equation 4.2)

where:

R	the radius of curvature
Factor(PCA)	the cracked status of the previous surface layer (0 means false, 1 means true)
AGE2	surface age
Surfnum	number of surface layers
ESA	average equivalent standard axles per day
SI_{rut}	structural rutting index
Kci_{OGPA}	the calibration coefficient for OGPA

Figure 4.8 illustrates some unrealistic results. While the average life expectancy of OGPA surfaces is in the order of 8 years, according to experience on the Auckland motorway, the figure suggests that an overlay on an uncracked surface will exhibit crack initiation at an age of 10 years, and that a new surface will last for approximately 20 years before it cracks.

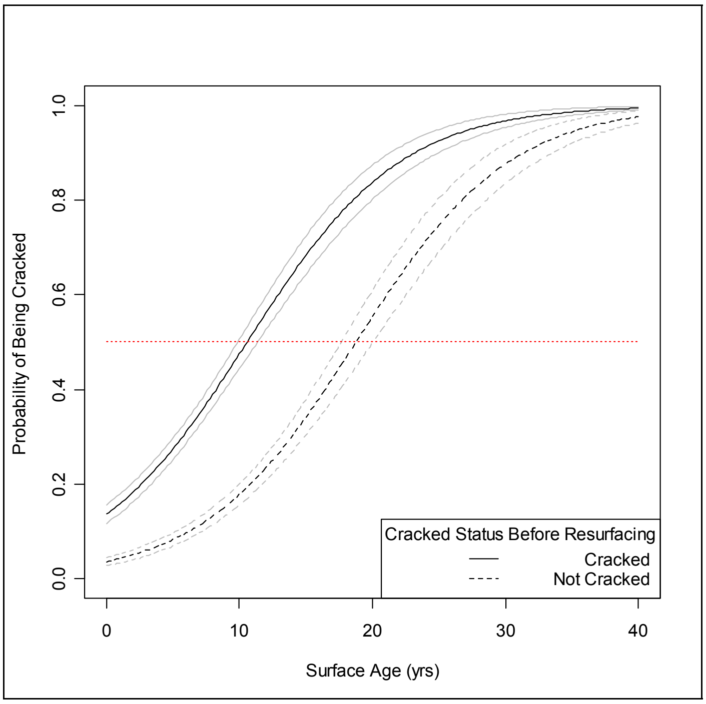


Figure 4.8 Predicting crack initiation for OGPA surfaces

Figure 4.9 confirms the poor predictive power of the crack-initiation model on the OGPA surfaces. It suggests only a 15 percent correlation between the model and the actual crack initiation.

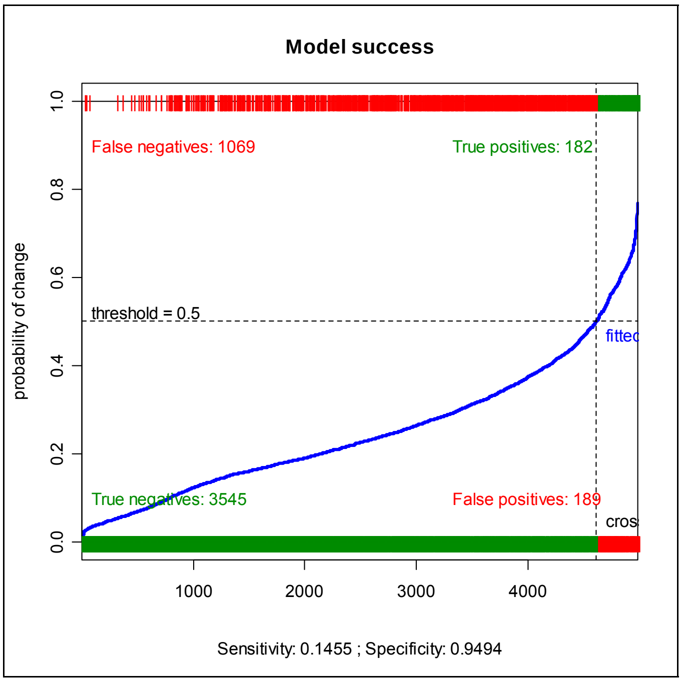


Figure 4.9 Testing the success of the OGPA surface crack initiation model

Based on the findings in this section, adoption of the crack-initiation model for AC surfaces is recommended, but not the model for OGPA surfaces.

5 Ravelling of OGPA surfaces

5.1 Objectives of the ravelling model

Section 2.2 illustrated that ravelling is one of the primary failure mechanisms of OGPA surfaces. Consequently, it is one of the primary drivers of maintenance decisions on the motorway networks. Ravelling in itself may start as a small defect, but it is an indicator of more serious defects to follow. Secondary defects include a higher degree of ravelling, delamination and potholes.

For this reason, the first occurrence of ravelling of OGPA surfaces would trigger an overlay, or mill and replace, within one year. Without a doubt, the modelling of ravelling on OGPA surfaces is essential in the maintenance decision support system for modelling motorway networks.

Taking account of ravelling in the maintenance planning process resulted in the following objectives for this development:

- The resulting model/s must be able to predict the first occurrence of ravelling on OGPA surfaces.
- It must incorporate input data that is readily available from network level databases.
- Consequently it must be able to predict the performance of in-service pavements.
- Any shortcomings in the current maintenance regime of OGPA surfaces should be highlighted through the model – i.e. the model must be able to simulate level of service for various maintenance options.

The following sections discuss the results of the ravelling model.

5.2 Exploratory statistics

Figure 5.1 illustrates the distribution of ravelling initiation. It is clear that the majority of all surfaces are prone to ravelling early in the life of the surface. As indicated in the appendix, the 75th percentile of all surfaces would ravel at an age of 3 years. There are some doubts as to the validity of this observation, as true ravelling of OGPA surfaces can only be identified by close inspection of perceived ‘missing chips’ within the open structure of OGPA surfaces. Having said that, there is a real possibility of OGPA surfaces ravelling at an even earlier stage, whilst stabilising later as chips settle into an optimal matrix. There are also questions regarding the RAMM rating procedure, especially on motorway sections such as those included in this research.

Therefore, any resulting model from this research needs to be thoroughly calibrated to any region in New Zealand, in order to ensure that it predicts the actual occurrence of ravelling.

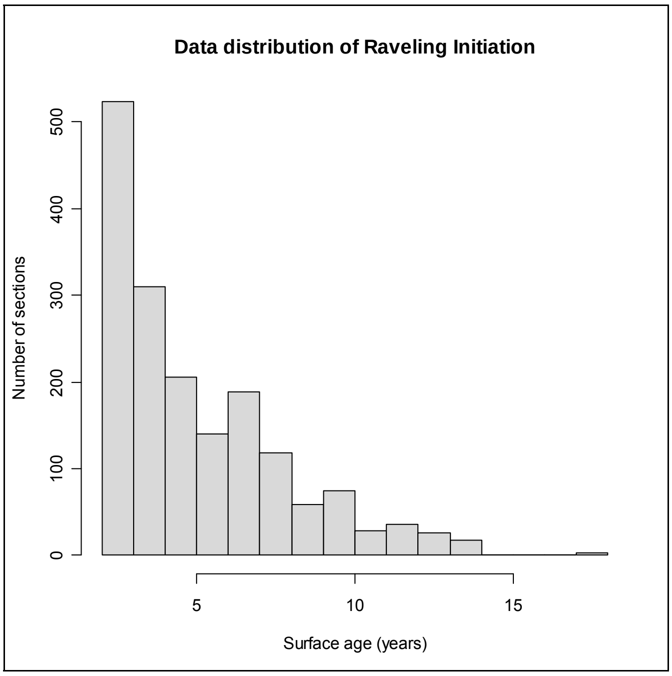


Figure 5.1 Distribution of ravelling initiation

The ravelling initiation was plotted as a function of the traffic loading (similar to figure 5.2) and some of the observations include:

1. There is a positive relationship between ravelling initiation and the traffic loading. This observation is against intuitive expectations, as one would expect roads with a higher traffic volume to ravel earlier. This trend is discussed further in the following paragraph.
2. The ravelling was plotted against both the equivalent standard axles per day and against the number of equivalent light vehicle axles. The heavy traffic loading effect was more prominent in predicting the occurrence of ravelling.

The next step was to further investigate the positive trend between ravelling and traffic loading, as presented in figure 5.2.

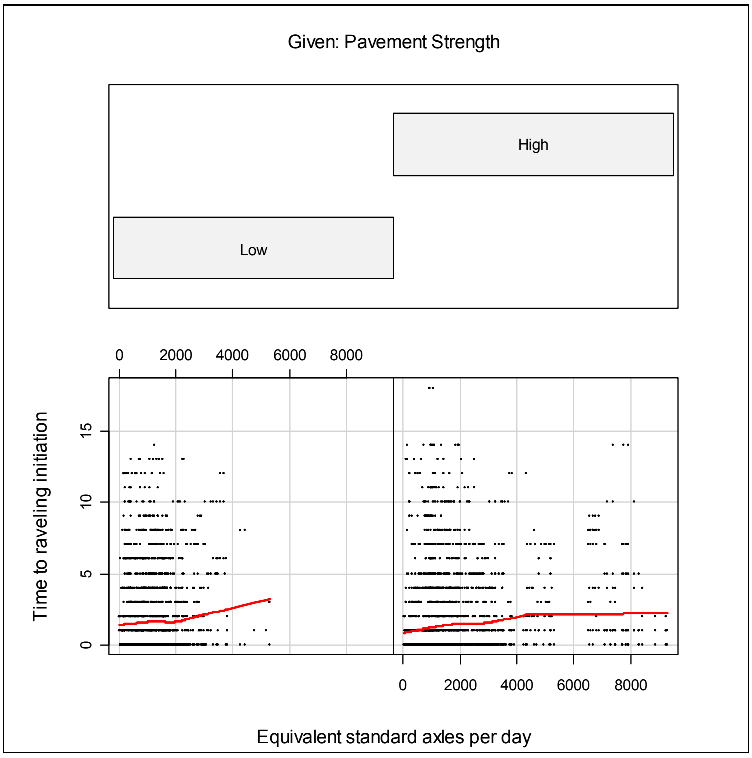


Figure 5.2 Ravelling initiation as a function of traffic and pavement strength

Figure 5.2 illustrates the combined effect of traffic loading and pavement strength in predicting ravelling initiation. The pavement strength was categorised in low strength for pavements having a SIrut less than 4, and high strength for more than 4. It is observed that there remains a positive trend between the traffic loading and the ravelling initiation, regardless of the pavement strength. However, this trend is significantly stronger for lower strength pavements. This suggests that for weaker/more flexible pavement, OGPA surfaces will ravel faster if the traffic loading is higher. More research is required to explain this observation.

The combined impact of pavement strength and the radius of curvature have been tested for a potential relationship with ravelling initiation. Figure 5.3 shows the outcome of this investigation – no clear trend was observed. However, it does appear that stronger pavements (higher radius of curvature) will delay ravelling initiation.

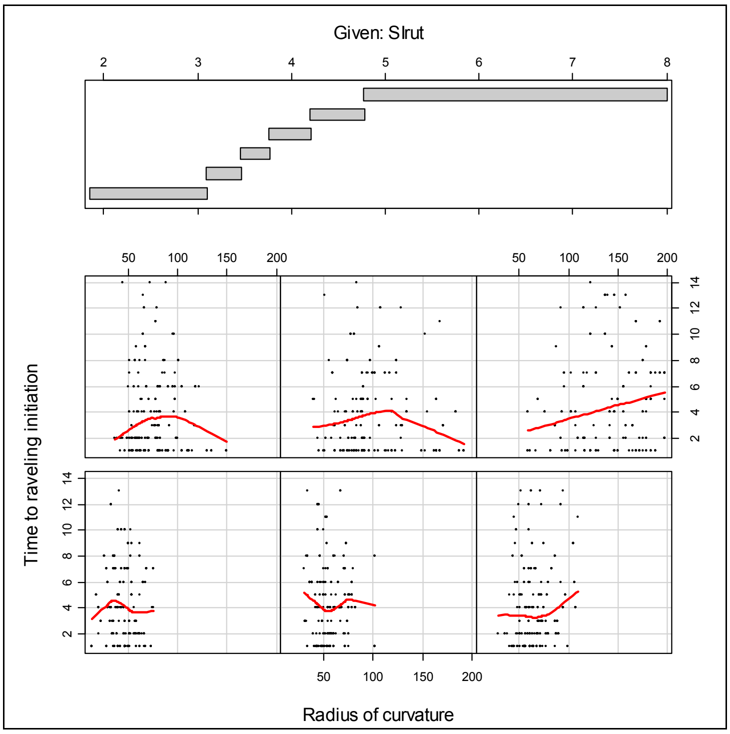


Figure 5.3 Ravelling initiation as a function of radius of curvature and structural rutting index

A potential correlation between the combined impact of the total surface thickness and time until crack initiation was tested – see figure 5.4. This figure demonstrates a very strong trend between crack initiation and ravelling initiation that is independent of the total surface thickness.

The result from the exploratory statistics has been useful in understanding ravelling in OGPA surfaces, and this understanding has been incorporated in the regression process, presented in the next section.

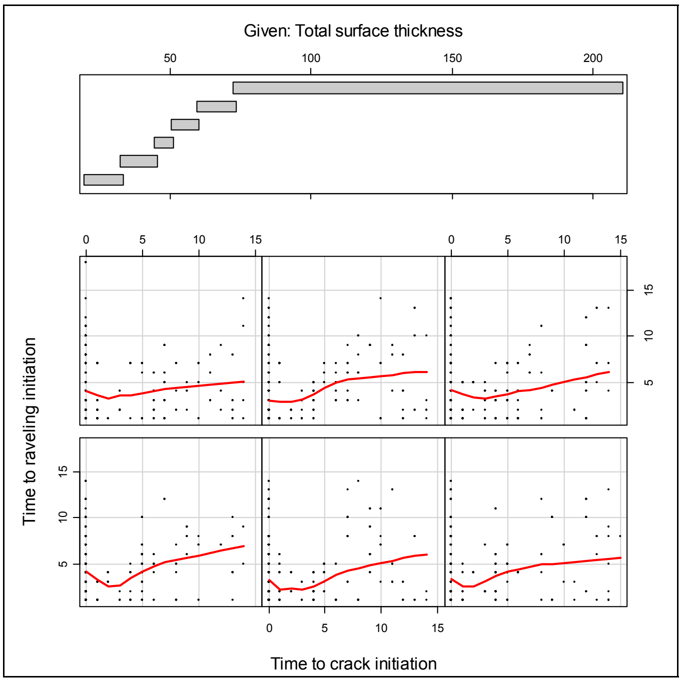


Figure 5.4 Ravelling as a function of crack initiation and total surface thickness

5.3 Predicting ravelling for OGPA surfaces

The outcome of the regression process is summarised in table 5.1 and equation 5.1.

Table 5.1 Regression outputs for OGPA ravelling initiation

	Estimate/ coefficient	Std. error	z value (sample variance)	Pr(>|z|) (confidence interval)	Significance
(Intercept)	-2.80123	0.309105	-9.062	< 2e-16	***
Age2	0.236933	0.009275	25.546	< 2e-16	***
Factor(crack_bln)1	1.359202	0.08988	15.122	< 2e-16	***
SI_{rut}	0.029524	0.026097	1.131	0.25792	
log(ESA)	0.139152	0.043803	3.177	0.00149	**

Significance codes:

'***' = 0, '**' = 0.001, '*' = 0.01, '.' = 0.05, ' ' = 0.1,

$$P_{rav_{OGPA}} = 1 \Big/ \left[1 + \exp\left(-0.237\,AGE_2 + Krav_{OGPA} \begin{Bmatrix} 2.801 - 0.0295\,SI_{rut} - \\ 0.139\,Log(ESA) - \\ 1.359\,CrackStatus(0,1) \end{Bmatrix} \right) \right]$$

(Equation 5.1)

where:

PravOGPA	the probability of an OGPA surface to be ravelled
AGE2	surface age
ESA	average equivalent standard axles per day
SI_{rut}	structural rutting index
Cracked status	whether a surface is currently uncracked (0) or cracked (1)

It is observed that there are high correlations with most factors, except for the structural rutting index/pavement strength. However, it was considered that this factor is prominent in the performance of OGPA surfaces and therefore it was retained for the expression.

Figure 5.5 and figure 5.6 present the outcome of the final model. Figure 5.5 illustrates the predicted model for a typical OGPA surface and pavement combination found on the dataset. It is observed that once a surface is cracked, ravelling would occur almost immediately. We suspect that the cracking observed on OGPA surfaces and ravelling could well be part of the same failure mechanism.

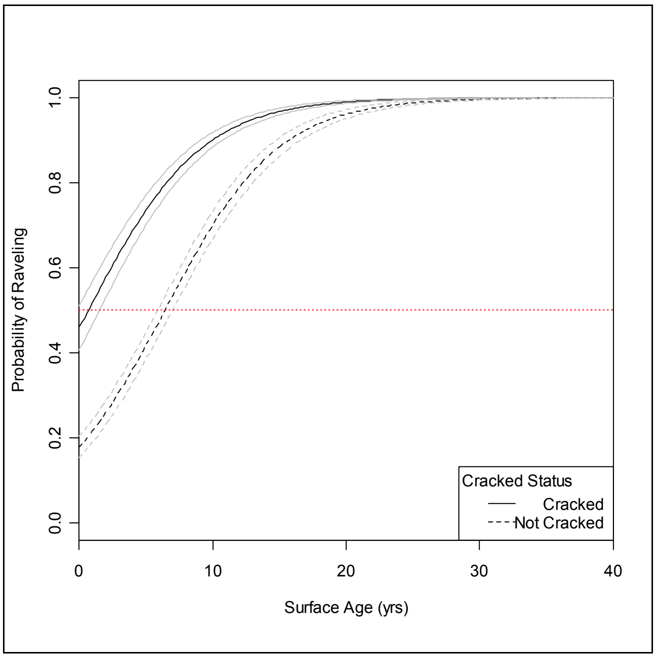


Figure 5.5 Predicting ravelling initiation for OGPA surfaces

Figure 5.6 illustrates the success of the model's prediction. It is observed that this model has an agreement of up to 75 percent with the actual behaviour, which is an extremely satisfying outcome. It is therefore recommended to adopt the model into the New Zealand dTIMS system.

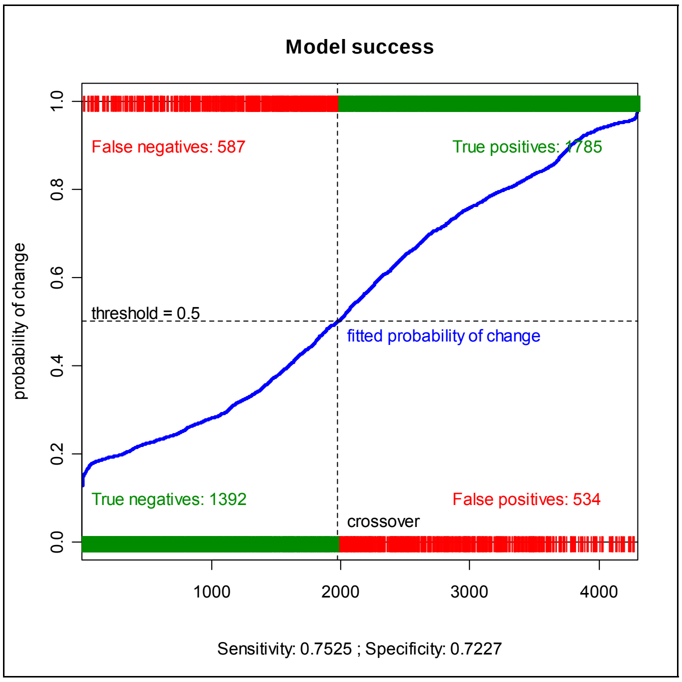


Figure 5.6 Testing the success of the OGPA surface ravelling initiation

6 Predicting rutting for AC surfaces

6.1 Rut change over the life of pavements

There is an expectation that rutting on pavements increases over time. This has been confirmed by other research, and is especially prevalent on thin, flexible pavements. (Theyse et al. 1996; Alabaster and Fussell 2006). However, this study indicated that on thicker asphalt-surfaced pavements, rutting only marginally increases over time.

Figure 6.1 illustrates the change in rutting with cumulative traffic loading. It is observed that there is only a minor increase in rutting in the later years of the pavement's life. It is also observed that there is a significant variance in the incremental rutting, which ranges from -4 mm to 4 mm per year. These variances are mostly explained in the high-speed data measurements, and this is also the reason for relatively poor regression results on the network data.

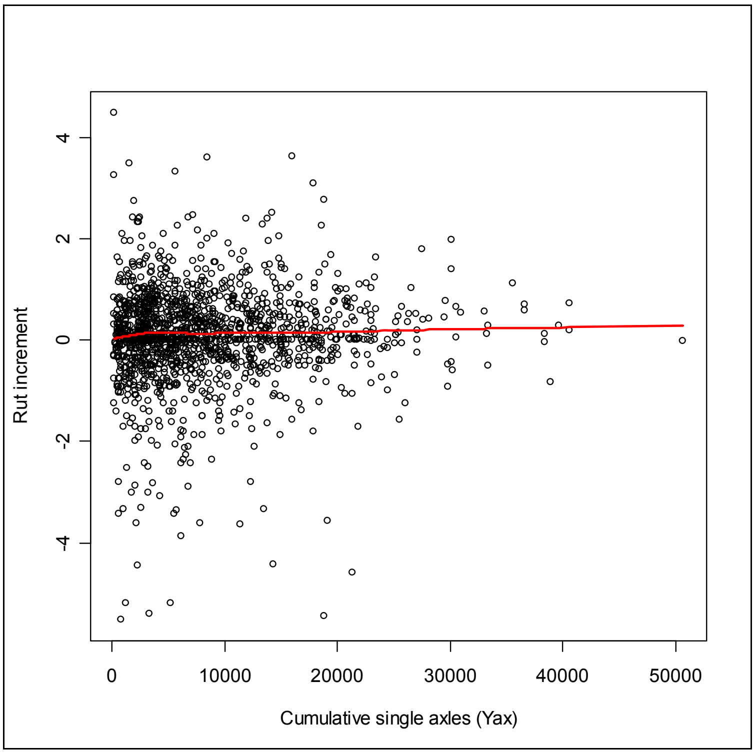


Figure 6.1 Change in rutting for cumulative traffic loading

Figure 6.2 investigates the different rut rates for variable pavement strengths. As expected, it seems there is a potential relationship between these factors – weaker pavements tend to have higher rut change for comparable traffic loading.

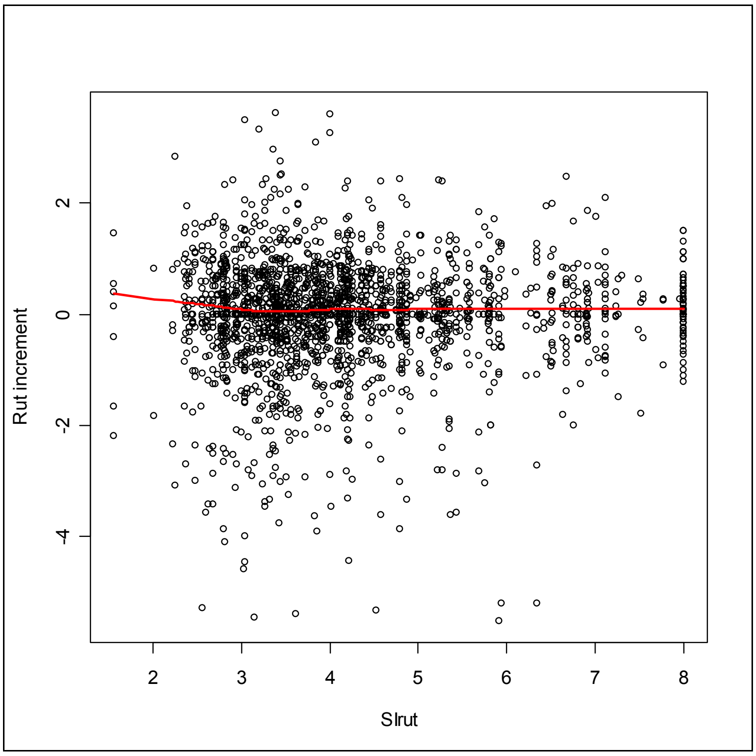


Figure 6.2 Rut increments for different pavement strength indices

6.2 Regression results

No satisfactory result could be obtained for rutting on the given dataset. Consequently, it is recommended that the rutting model structure, as reported in Section 2.3, should be retained and refined according to individual asphalt LTPP sites.

7 Summary

7.1 Outcomes achieved

7.1.1 New models

The objective of this research was to develop pavement deterioration models for application on asphalt pavements. In addition, it aimed to differentiate between the performance of dense-graded asphalt and porous asphalt (OGPA).

The research successfully achieved the following:

- development of an empirical-probabilistic model that predicts the likelihood of cracking for a dense-graded asphalt pavement
- the use of this model to forecast the probability of an OGPA pavement to ravel

This research also attempted to develop a crack-initiation model for OGPA surfaces, but the results were not satisfactory. We suspect this was because of some data issues, and it was not possible to develop a robust model.

7.1.2 Model testing

A new technique was adopted to test the accuracy/validity of the newly developed models. An example of the output from this technique is presented in figure 7.1.

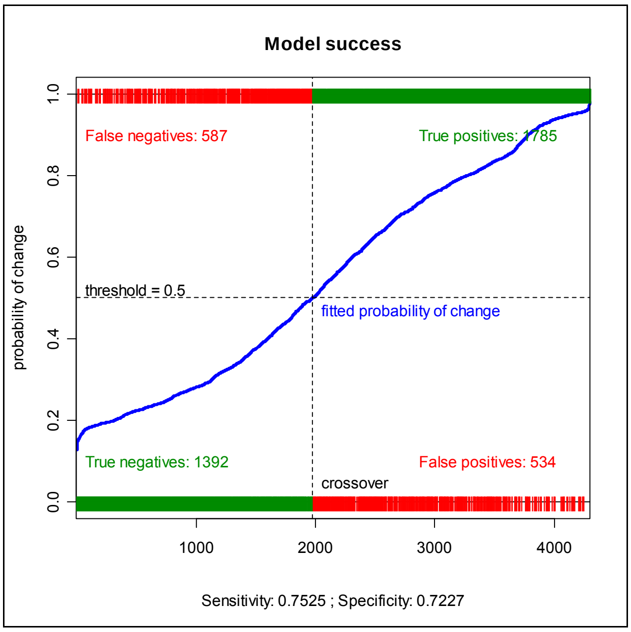


Figure 7.1 Testing the accuracy of the ravelling model for OGPA surfaces

This figure illustrates the accuracy of the OGPA ravelling model. It validates true and false predictions for cracked and uncracked surfaces by comparing the model outcome to the actual status of surfaces.

In this instance, it is observed that the model is correctly predicting 75 percent of the sections. This is an exceptionally high correlation for pavement models.

Based on this technique, both the models for crack initiation on AC and ravelling on OGPA surfaces were accepted for adoption within the national dTIMS system for New Zealand roads.

8 Recommendations

8.1 Techniques for adopting the models

There are two options available for the adoption of the probabilistic models developed for this research:

1. Directly use the models in their probabilistic format.
2. Transform the probabilistic model into an absolute model, accepting initiation at 0.5 probabilities.

Our experience with both these methods is discussed in the following sections.

8.1.1 Using models directly in probabilistic format

There are two applications for using the initiation model in its probabilistic format.

8.1.1.1 For intervention

The model can be used in identifying rehabilitation or maintenance actions, commonly referred to as trigger levels. Figure 4.6 illustrates the predicted crack initiation for dense-graded asphalt surfaces. In this figure, the 0.5th probability is indicated as the point in time when cracks would be expected to appear – maintenance or resurfacing treatment should be considered soon after that, in order to keep the surface watertight.

This point of intervention would be a function of the level of service, criticality or importance of a road and the traffic it carries. By using the model in its probabilistic format, the probability of intervention can be varied. For example, in areas of normal operating conditions, a crack probability of 0.7 might be the intervention point, but on more important routes, this might be set at a crack probability of 0.6.

8.1.1.2 As a reporting tool

The power of the probabilistic model is actually realised in the reporting of the condition probability distribution of the network over time. The figure on the next page shows an example of the cracking probability of an entire network. The probability has been classified into five categories, and the percentage of the network within each of these categories is shown for the next 10 years.

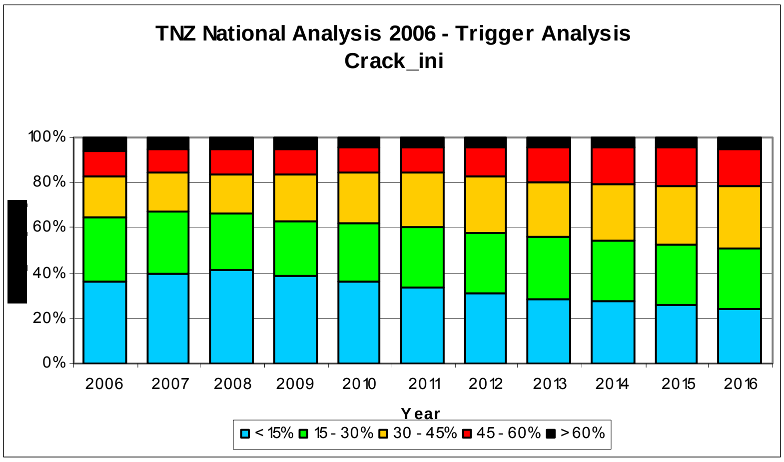


Figure 8.1 Probability of cracking due to decreased funding levels (Transit 2007)

From figure 8.1, it is clear that for this particular network and maintenance regime, the length in the category of 'very low risk for cracking' (<15%) has halved for the next ten years. The 'high' and 'very high risk' categories (45<%) have significantly increased. It can therefore be concluded that this particular maintenance regime is unable to maintain the current risk level on the network.

8.1.2 Transforming the model into an absolute function

It is recognised that the industry will take some time to adjust to the new model formats. For this reason, the second adoption technique is to transform the model into an absolute model. This is achieved by replacing the probability of the model at 0.5 and using the age term (AGE2) as the dependent variable. Using this format will still yield the expected time of crack initiation in terms of years.

Although this technique still provides the user with a familiar outcome, it limits the power of the model by setting the intervention period based on a fixed crack-initiation time.

8.2 Further research

Although this research had some excellent outcomes, it is realised that the model development is subject to constant refinement. In addition, it is further recognised that implementing the new models into the New Zealand dTIMS system will highlight further research needs.

Based on the above, and some practical findings of this research, the further work required is summarised in table 8.1.

Table 8.1 Summary of further research work

Research area	Description of further work
Model refinement	Once more appropriate data is available, the models needs to be refined, including the: • rutting model for asphalt and chipseal pavements • crack-initiation model for OGPA surfaces.

Practical aspects	This research, along with earlier Land Transport NZ research, has highlighted a number of performance and costs issues related to the overlay practices used on OGPA pavements. Best-practice guidelines should be developed that will ensure life-cycle cost efficiencies in these practices.
Including variable section lengths	This research only considered fixed inspection lengths according to a 10% sample. More research is required to investigate the influence of section length on the defect probabilities.
Model adoption	The adoption of these new models into the dTIMS system will also require changes to the decision logic for the system. These changes need to be documented and published with other dTIMS publications.
Further asphalt data needed	Most of the developments in this research had to rely on network data because the LTPP data on asphalt pavements were too limited. The expansion of this programme to include more asphalt pavements is essential.

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Appendix Descriptive statistics

Network sections		Structural number (snp_mech)		Annual daily traffic (ADT)	
AKL	2518	Min.	1.560	Min.	9.42
WEL	2450	1st Qu.	3.290	1st Qu.	3338.99
		Median	3.900	Median	8228.70
		Mean	4.251	Mean	10767.41
		3rd Qu.	4.910	3rd Qu.	15999.50
		Max.	8.000	Max.	84234.84
		NAs	1672.000	NAs	131.00
Percent cars (pc_car)		**Percent heavies (pc_hcv)**		**Surface depth prior to new surface (hold)**	
Min.	63.07	Min.	0.700	Min.	10.00
1st Qu.	84.00	1st Qu.	3.990	1st Qu.	37.75
Median	89.00	Median	4.330	Median	51.00
Mean	87.33	Mean	5.146	Mean	60.49
3rd Qu.	91.00	3rd Qu.	6.410	3rd Qu.	66.00
Max.	97.00	Max.	18.000	Max.	318.00
NAs	214.00	NAs	214.000		
New surface age (hnew)		**Total surface thickness (htot)**		**Surface date (surface_date)**	
Min.	10.00	Min.	20.00	20/2/2000	90
1st Qu.	25.00	1st Qu.	63.00	1/4/2004	83
Median	30.00	Median	83.00	29/4/2003	70
Mean	35.63	Mean	96.12	7/8/2004	70
3rd Qu.	35.00	3rd Qu.	100.00	1/5/2003	67
Max.	300.00	Max.	618.00	25/12/1997	67
				(Other)	4521
Binder type (surf_binder_hist)		**Ravelling**		**Cracking**	
(Blank)	2650	Min.	0	Min.	0.0000
B80	1460	1st Qu.	0	1st Qu.	0.0000
B60	344	Median	0	Median	0.0000
B180	291	Mean	0.02069	Mean	0.2499
E80	127	3rd Qu.	0.0002897	3rd Qu.	0.5000
Unknown	72	Max.	1	Max.	1.0000
(Other)	24	NAs	1648	NAs	1623.0000
Mean profile pepth (MPD)		**Rut depth**		**Rut depth standard deviation**	
Min.	0.180	Min.	0.000	Min.	0.170

1st Qu.	0.610	1st Qu.	2.140	1st Qu.	0.930
Median	0.920	Median	2.870	Median	1.280
Mean	1.021	Mean	3.116	Mean	1.385
3rd Qu.	1.300	3rd Qu.	3.835	3rd Qu.	1.700
Max.	4.080	Max.	22.420	Max.	10.880
NAs	853.000	NAs	853.000	NAs	1338.000
Roughness (iri)		**Surface age (Age2)**		**Pavement age (Age3)**	
Min.	0.89	Min.	0.000	Min.	1.00
1st Qu.	2.46	1st Qu.	4.000	1st Qu.	16.00
Median	3.15	Median	7.000	Median	24.00
Mean	3.35	Mean	7.644	Mean	26.58
3rd Qu.	4.05	3rd Qu.	11.000	3rd Qu.	36.00
Max.	35.48	Max.	38.000	Max.	79.00
NAs	853.00				
Total number of surfaces (Numsurfgrow)		**% Cracking before resurface (PCA)**		**Crack initiation (crackini)**	
Min.	0.000	Min.	0.0000	Min.	0.000
1st Qu.	2.000	1st Qu.	0.0000	1st Qu.	0.000
Median	3.000	Median	0.0000	Median	0.000
Mean	3.017	Mean	0.4064	Mean	2.297
3rd Qu.	4.000	3rd Qu.	1.0000	3rd Qu.	4.000
Max.	7.000	Max.	1.0000	Max.	36.000
Ravelling initiation (ravini)					
Min.	0.0				
1st Qu.	0.0				
Median	1.0				
Mean	2.3				
3rd Qu.	3.0				
Max.	37.0				