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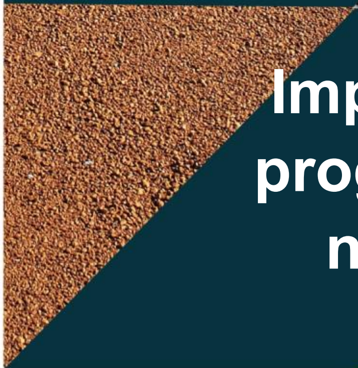
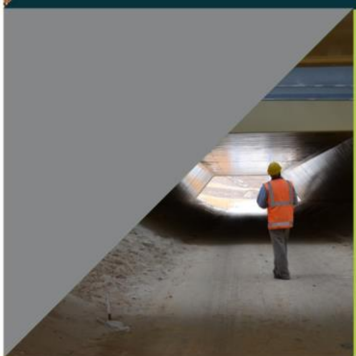
WESTERN AUSTRALIAN ROAD RESEARCH AND INNOVATION PROGRAM



Improving decision making and works program development with continuous network strength and condition data

Stage 1 Final Report



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mainroads WESTERN AUSTRALIA

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Improving decision making and works program development with continuous network strength and condition data

for Main Roads Western Australia

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PRP17024-1
June 2019

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IMPROVING DECISION MAKING AND WORKS PROGRAM DEVELOPMENT WITH CONTINUOUS NETWORK STRENGTH AND CONDITION DATA

VERSION CONTROL					
ARRB Project No	PRP17024		Client Project No		
Path	c:\users\elsabe.vanaswegen\desktop\edited warrip 038 - improved decision making - final submission-tc.docm				
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Task	Date	Technical/Quality Checks	Responsibility	By (Initials)
1	21/08/2018	Initial Draft using required and approved template and following reporting formatting guidelines	Author	LN
2	21/08/2018	Spell checked	Author	LN
3	21/08/2018	All tables and figures/images checked for source and permission for use (where appropriate/applicable)	Author	LN
4		References and superseded references checked by Library, Library comments addressed, updated ref list pasted into report	Author	
5	23/08/2018 and 22/05/2019	Checked by QM for technical rigour and accuracy (discuss required changes with Author)	Quality Manager	TYT
6	22/05/2019	QM to send report to editor@arrb.com.au	Quality Manager	TYT
7	08/06/2019	Report reviewed for formatting, grammar, use of referencing and overall presentation and return to Quality Manager	Editor	PM
8	10/06/2019	QM addresses Editor comments (liaising with Author)	Quality Manager	LN
9	10/06/2019	QM to send final report to ARRB PL, Program Manager and cc. Author and Library	Quality Manager	LN
10	12/06/2019	Release to client with a cc. to the Author, QM and editor@arrb.com.au	Program Manager	EvA
COMMENTS				

SUMMARY

The project set out to maximise the use of the pavement strength data collected with the Traffic Speed Deflectometer (TSD) in 2016 over approximately 900 km of the MRWA sub-network, the TSD 900. The survey was conducted by ARRB and led to the provision of TSD survey output which utilised the area under the curve method to estimate deflection parameters. These were then converted to an equivalent Falling Weight Deflectometer (FWD) deflection using a relationship developed from research in Queensland.

A review of international and domestic practices on determining pavement structural capacity was carried out. Three methods of determining a pavement structural index were identified and compared using the data from the TSD 900. Two variations of Method A, the Austroads structural number (SNC) method, Method B being an adaptation of the Austroads rehabilitation design method to determine a Notional Structural Life, and Method C which utilised the ARRB Structural Evaluation of Pavement (STEP) method were compared.

The Remaining Structural Life (RSL) calculated with Method A which estimated the initial Modified Structural Number after construction (SNC_0) and Method C fluctuated considerably with a change in deflection and pavement type. The estimated RSL was shorter for both methods when the base material was stabilised. The shortest RSL was obtained when the pavement type is a thin asphalt on a stabilised pavement. Method A with the back-calculated SNC_0 and Method B produced a constantly high value of RSL with the former not affected by fluctuations in measured deflections.

Environmental and geological factors were investigated to assess their influence on structural deterioration, with the Thornthwaite Moisture Index (TMI), temperature and precipitation used as the environmental factors and with the geological factor represented by the WA soil classification. A statistical correlation analysis was investigated with the RSL from Method C selected as the dependent variable. None of the factors were found to have a significant negative correlation. However, precipitation was found to produce a better correlation than TMI or temperature.

A suite of case studies was carried out by applying the TSD data in a pavement management system environment using Deighton's Total Infrastructure Management System (dTIMS). Five combinations of the ARRB dTIMS setup which employed three different structural index methods were developed and applied within the existing ARRB dTIMS template which utilises the Austroads functional road deterioration (RD) and works effects (WE) models. The treatment rules and triggers employed generally followed those specified by Main Roads Western Australia (MRWA) with one exception of replacing the deflection and curvature parameters in the MRWA setups with RSL. MRWA separately ran their own dTIMS setup to generate comparable results. The analysis results were compared by reviewing the length and type of treatments triggered over the 20–year analysis period, and treatment costs.

- The analysis results of the Austroads SNC method (ARRB_A2) and Notional Structural Life method (ARRB_B2) were almost identical in terms of the total cost as well as when treatments are triggered, and are not substantially affected by the structural deterioration during the analysis period
- The lowest treatment cost was the MRWA setup, with this giving priority to preservation treatments (reseal/resurfacing) over rehabilitation.
- The setup that produced the costliest work program over the 20–year period is the ARRB STEP Procedure (ARRB_C2).

Recommendations from the study include the following:

1. There is a significant difference in the roughness and rutting model estimates from applying the Austroads/ARRB and the MRWA models as reflected in the analysis results. This should be addressed through a comprehensive calibration exercise based on time-series data to ensure the models match the actual network performance[1].
2. A wider sample, including other pavement configurations covering a broader range of climate zones and other factors such as drainage condition, pavement age and updated soil information should also be investigated.
3. Whilst roughness and rutting have been well integrated into MRWA pavement modelling, cracking data is not yet fully utilised. MRWA should investigate adopting an incremental cracking model such as the Austroads cracking model instead of relying on annual crack scores assessed by the region.
4. The immediate validation by means of field investigation of the analysis results using the first-year work programs is highly recommended to confirm the accuracy of the setups. However, to assess the prediction of future needs, a combination of a calibration exercise using historical condition data and a long-term monitoring program is recommended. With the TSD, MRWA is now able to obtain more precise functional and structural data and should take advantage of its availability.
5. Further work should aim to enhance the current MRWA dTIMS setup by taking advantage of the finer detail from the TSD data to identify potential structural issues, i.e. 'pavement repairs' as input to works programming and costing.
6. The conversion of TSD measured deflection to the FWD equivalent should use the relationship from the Western Australia study when it is available.

[1] Since the completion of this study MRWA has re-estimated road deterioration models for WA, and the results are reasonably consistent with the Austroads/ARRB models. The implication of this is both budget and condition estimates should therefore be similar, but this can only be confirmed by rerunning the MRWA dTIMS setup.

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TABLES

FIGURES

1 INTRODUCTION

1.1 Background

MRWA has embraced and led the development and application of sound asset management processes over many years. With the introduction of new data collection technology and with MRWA having initiated a review of current MRWA systems and processes in the complementary WARRIP project 'Towards Best Practice in Management of Road Pavement Assets Stage 1' (herein referred to as the *Towards Best Practice* project), it is an opportune time to explore further improvement of processes and decision making related to road network investment.

By undertaking this project, (herein referred to as the *Improved Decision-Making* project), in parallel with the *Towards Best Practice* project and outlining recommendations for improvements, MRWA will be best placed to take advantage of the investment it is making in the acquisition of continuous network strength and condition data provided by the ARRB Traffic Speed Deflectometer (TSD).

Given the complementary nature of these two projects, it is intended that both projects be undertaken in two stages, with the second stage of each project being more fully scoped after considering the outcomes of stage one of the sister project. That is, Stage 1 of the *Improved Decision-Making* project ran in parallel with Stage 1 of the *Towards Best Practice* project. In this regard, MRWA will have an opportunity to ensure that the focal point of this project, to be ready for the delivery of the soon to be acquired high speed deflection and surface condition data, will be well considered with detailed knowledge and mapping of what it is currently doing in pavement asset management more broadly.

1.2 Project objectives

The objective of the project was to undertake a state of practice review of the use of continuous strength data in pavement maintenance decision making.

1.3 Project scope and output

The tasks to be undertaken in this project have been split over 2 stages, and Stage 1 which is the subject of this report includes the tasks shown in Table 1.1 aimed at providing a status assessment and proposed improvements to MRWA processes and tools. A final output involves undertaking a workshop with key MRWA staff to hand over outcomes of Stage 1 and agree on the scope of Stage 2.

Table 1.1: Project tasks

Task	Sub-task	Milestone/task description
1	1a	Review of available methods (domestically and internationally) for determining and estimating a pavement structural index
	1b	Review of the impact that environmental and geological features have on pavement performance
	1c	Review of the suitability of other required parameters in the modelling process
	1d	Development of case studies using available TSD data

Task	Sub-task	Milestone/task description
2	2a	Drafting of report to outline proposed development considerations sent to MRWA
	2b	ARRB to address MRWA feedback and submit final report to MRWA
	2c	Undertake workshop to discuss findings in the report, explore other areas of further investigation and agree on the scope of Stage 2 (Location: Perth)

The report is structured as follows:

- Section 2, *Data Assembly*, details the assembly of the dataset for a few demonstration tasks under the project. An explanation is also provided on how the data from the trial 900 km of TSD survey (TSD 900) was prepared and converted to FWD equivalent deflections.
- Section 3, *Review of methods for determining a pavement structural index*, describes a selection of international and domestic practices used to identify structural needs (and weak pavements) and how they are used at a network and project level. A more detailed review of three selected methods deemed relevant for adoption by MRWA is also discussed including a comparison of the methods by applying them to the TSD 900.
- Section 4, *Review of factors influencing structural deterioration*, considers the climatic and geological features which have been identified as potential factors along with others such as pavement age, temperature and level of precipitation.
- Section 5, *Case studies*, describes practical examples of applying the TSD data in a pavement management system environment. This section also explains the configuration and models used in dTIMS as well as comparing the analysis results produced by the MRWA-dTIMS and the ARRB-dTIMS setups for the project.
- Section 6, *Conclusions and recommendations* summarises the study findings and their proposed application, and possible further research.

The report also includes the following appendices:

- Appendix A, Summary of data received
- Appendix B, ARRB treatment trigger example for AW road class
- Appendix C, Soil classification for the project
- Appendix D, Dynamic segmentation tool

The report is also accompanied by two files developed and used in the project, which comprise:

- the dynamic segmentation tool employed in the project which is embedded in a Microsoft Access file
- the visualisation tool employed to compare the three pavement structural indexing methods which is on a Microsoft Power BI platform.

It complements the final report from the WARRIP project '*Towards Best Practice in Management of Road Pavement Assets Stage 1*' (Toole 2019).

2 DATA ASSEMBLY

2.1 The TSD 900 data

In 2016, ARRB surveyed approximately 900 km of the MRWA road network (TSD 900) with the TSD survey vehicle. The trial route extended approximately from Eucla at the Western Australia – South Australia border then westwards to Perth including a sample of four major roads around Perth. A typical TSD survey collects functional condition via laser profiler, automatic crack detection and high definition video, and structural information reported as an estimated deflection bowl based on a model developed by Muller and Roberts (2012).

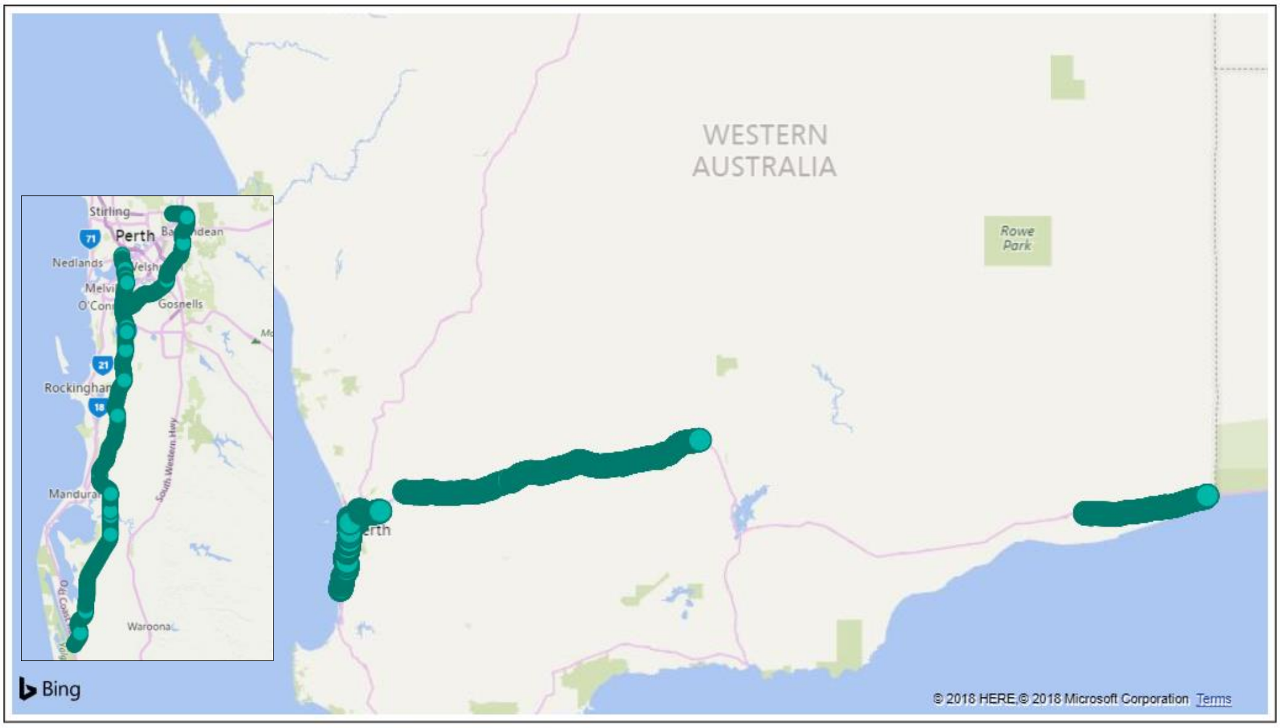
In general, data was available at a lane level i.e. for a single carriageway (code S) with both lanes surveyed. For a dual carriageway (code L for the prescribed direction and R for the opposite) only one lane in each carriageway was surveyed.

For this study, the total surveyed length of 1,447 lane km is listed by road number in Table 2.1, and the geographical location of the sections is illustrated in Figure 2.1. For a detailed start and end SLK of the survey extent, refer to the accompanying visualisation tool.

Table 2.1: Extent of the 900 km TSD trial survey

IRIS_ROAD_NO	RUN_DIRECTION	Length by carriageway type (km)		
		L	R	S
H003	L			181.1
H003	R			181.1
H005	L	29.7		454.7
H005	R		29.6	292.7
H015	L	57.2		
H015	R		71.7	
H018	L	34.4		
H018	R		34.3	
H021	L	2.6		
H021	R		2.6	
H057	L	37.7		
H057	R		37.7	
	Total	161.5	175.8	1,109.5

Figure 2.1: Map showing the TSD 900 survey



Much of the data used on the project was sourced from the results of the TSD 900 from ARRB and the MRWA's IRIS data repository, whereas the remaining data was obtained from several other sources. The data came in various formats and levels of detail, with most requiring some form of data manipulation before application. The complete list of data received, and its use is provided in Appendix A.

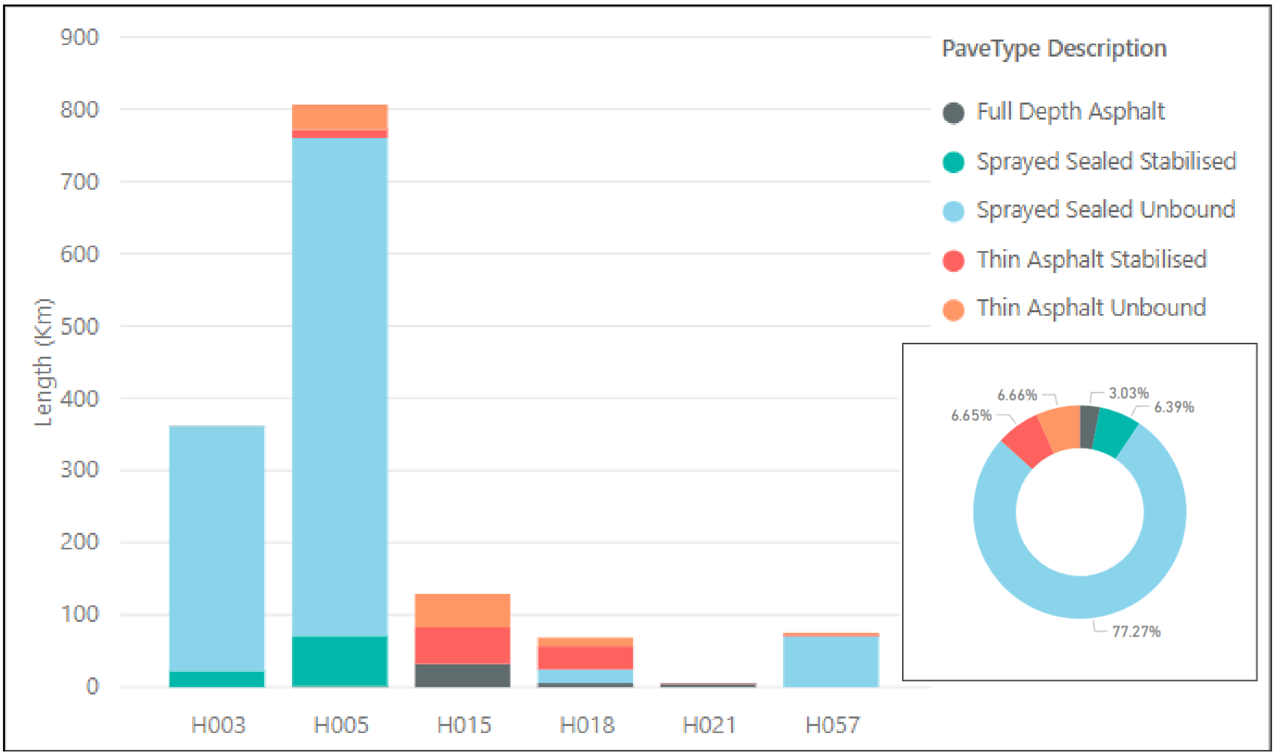
The data supplied by ARRB did not align fully with the MRWA network reference system and a geo-referenced exercise was therefore undertaken to ensure alignment. The advantage of using the ARRB TSD dataset was because it provided the cracking information as well as the other functional condition information and the structural data.

The TSD 900 mostly consists of two-lane, two-way unbound granular pavement with a thin bituminous (sprayed seal) surfacing. This applies to the rural portion of the trial route along the Eyre Highway (H003) and the Great Eastern Highway (H005). There is however, a small portion of full depth asphalt in and around the Perth area. Figure 2.2 details the configuration of the pavements that make up the various routes.

Once assembled the dataset was used at:

- 10 m intervals for input to a dynamic segmentation exercise
- aggregated 100 m intervals for comparison between the three selected structural indexing methods
- aggregated 500 m intervals as input to the MRWA and the ARRB dTIMS setups.

Figure 2.2: TSD 900 surface and pavement configuration



2.2 Using TSD measured deflection

As a reasonably new technology, a current dilemma amongst asset managers and pavement design practitioners is how to apply TSD measured structural data for planning and for design purposes. Established methodologies have been developed based on the principle of measuring strength by the Falling Weight Deflectometer (FWD) device, or by Benkelman Beam and related technologies such as the Australian adaptation of the deflectograph, e.g. the VicRoads PaSE (VicRoads' Pavement Strength Evaluator). These methods were predominantly developed for project level purposes and utilise either back-calculation methods to determine layer moduli as part of a mechanistic design procedure or chart-based methods, also referred to as empirical procedures. Both are described in Austroads (2012).

For network level asset modelling purposes, road performance relationships (Austroads 2010a) express pavement strength as a single number, termed the modified structural number (SNC) (Hodges, Rolt & Jones 1975). The SNC has been correlated with the Benkelman Beam and the FWD based on work by Paterson (1987).

At the time of writing, no method exists to use the measured deflection bowl from TSD as is for design or planning purposes.

The ARRB TSD survey output utilised the area under the curve method to calculate the deflection at a specified distance from the centre of the wheel (Muller & Roberts 2012).

A more recent study in this area was reported by Lee (2016) following research conducted under the Queensland DTMR/ARRB National Asset Centre of Excellence (NACOE) program. This study arranged for the TSD and FWD surveys to be undertaken at the same time, which in turn eliminated the variation of external factors that can impact the comparison results, such as temperature and subgrade moisture. As a baseline, movement sensors were also embedded in the surface to measure the actual deflection bowl. The output of the study was a relationship between the TSD deflection and components of the FWD equivalent bowl and produced the following relationship (Equation 1) which was used to estimate equivalent FWD deflections (measured in microns).

$$D_{0\text{-}FWD} = 0.9D_{0\text{-}TSD} + 138 \tag{1}$$

When comparing the TSD output against the FWD bowl and the instrumented actual bowl, the NACOE study concluded that beyond a distance of 300 mm from the theoretical loading plate, the result cannot be reliably correlated. Therefore, only the D_0 and D_{200} TSD results were used.

The following steps were taken in preparing the TSD data from the ARRB survey:

1. Synchronised ARRB survey chainage to MRWA reference SLK by means of geo-referencing. This enabled the use of the ARRB TSD dataset at 10 m intervals with other datasets supplied by MRWA.
2. Supply of TSD output components of the deflection bowl (D_0 and D_{200}) based on Muller and Roberts (2012).
3. Normalisation of the deflections to 10,000 kg based on the combined left and right strain gauge reading from the TSD survey.
4. Conversion of the normalised TSD deflections ($D_{0\text{-}TSD}$) to equivalent FWD deflections ($D_{0\text{-}FWD}$) using the NACOE relationship.

3 REVIEW OF AVAILABLE METHODS TO DETERMINE A PAVEMENT STRUCTURAL INDEX (TASK 1A)

3.1 General

As part of the tasks, the project team reviewed several methods of assessing pavement structural capacity, and how they might be used to inform a 'health index' from a structural perspective.

The range of methods available, including specific examples, and their typical application are summarised below, and a selection of these have been chosen for application in this project.

3.2 Range of methods

A number of alternative methods are commonly employed, including, the following methods:

1. Methods which employ empirical transient deflection versus pavement life relationships. Such relationships have been developed many years ago by TRL for use in the United Kingdom (Kennedy & Lister 1978) or have been adapted from design methods such as Austroads (2012) for use in Australasia, and by CSIR South Africa (1983). They have been widely applied in temperate, sub-tropical and tropical environments. The apparatus used includes Benkelman beams, the Deflectograph (such as PaSE) and the Falling Weight Deflectometer (FWD). A more recent example includes the ARRB-RMS structural evaluation of pavements (STEP) procedure (Loizos, Roberts & Crank 2002; Roberts 2017).
2. Certain pavement design methods, such as the AASHTO pavement design guide method (AASHTO 1993) which does not require surface deflection and is based on estimating the 'unconsumed' life of the original pavement.
3. Various mechanistic procedures which employ back calculation methods, i.e. methods which are a response to load, and utilise the results of FWD tests.
4. The family of incremental pavement condition models as represented by mechanistic-empirical deterministic models such as HDM-4 (Kerali 2000) and the Austroads network level deterioration models (Austroads 2010a; Austroads 2010b; Martin & Choummanivong 2018).

The main characteristics of each are summarised in Table 3.1, with comments made on their intended application, restrictions or limitations on use, data requirements and overall suitability.

Table 3.1: Alternative methods for determining the remaining pavement structural life

Characteristics	Incremental pavement deterioration models	Empirical deflection-life based methods	Pavement design-based methods	Mechanistic pavement design methods
Principal application	Life-cycle performance prediction of new and existing pavements, including post-treatment	Determination of strengthening needs to carry future traffic	Design of new pavements, or involving substantial replacement/reprocessing of bound and unbound layers	Design of new pavements and strengthening measures, including replacement/reprocessing of layers
Potential alternative application	▪ Estimation of time to stated condition limits (intervention levels), and condition states for functional and structural measures	▪ Determination of allowable loading for a selected treatment ▪ Remaining life of current pavement	▪ Determination of unconsumed life	▪ Determination of allowable loading for a selected treatment ▪ Remaining life of current pavement

Characteristics	Incremental pavement deterioration models	Empirical deflection-life based methods	Pavement design-based methods	Mechanistic pavement design methods
Typical restrictions on use, and stated deficiencies Other limitations	▪ Scope of original studies, including coverage of pavement types and treatments and key dependent variable, such as climate, traffic and other conditions ▪ Need for significant calibration effort, although possibility of auto-calibration using time-series data offers significant advantages	▪ Developed primarily for use in treating badly distressed pavements ▪ Potentially inapplicable to non-distressed pavements ▪ Maximum traffic levels ▪ Heavy-duty flexible pavements, which may show little change in deflection over time ▪ Particular failure modes, e.g. top-down cracking	▪ The large performance variations among identical designs ▪ Estimation of past ESAs ▪ Inability to account for pre-overlay repairs and sufficiently represent pavement strength ▪ Most applicable to pavements with very little visible deterioration.	▪ Limited to analysing the cumulative deformation of the whole pavement, related to the vertical strain in the subgrade, and asphalt fatigue. ▪ Interpretation of the deflection bowl has proved to be difficult because the assumptions of linear elastic theory do not hold sufficiently well.

Characteristics	Incremental pavement deterioration models	Empirical deflection-life based methods	Pavement design-based methods	Mechanistic pavement design methods
Data requirements	▪ General knowledge of pavement and surfacing type, and historical treatments ▪ Pavement functional condition data ▪ Representative or more detailed structural (deflection) data a significant advantage	▪ Closely-spaced measurements of D_0 and/or D_{200}, with prescribed standardisation and adjustments ▪ Pavement configuration and thicknesses, representing surface and principal load-bearing layers ▪ Future traffic loading	▪ Detailed knowledge of pavement structure, including layer types and thicknesses and their integrity ▪ Knowledge of past and future traffic	▪ Detailed knowledge of pavement structure, including layer types and thicknesses and their integrity, including layer moduli, etc.
Overall suitability	Highly suited to asset planning and monitoring purposes, with accuracy likely to improve with good quality data	Significant potential where combined with functional measures, noting key limitations	Too data hungry and time-consuming for network level analysis	Too complex and time-consuming for network level analysis
Example methods	Austroads functional and structural deterioration models	Adaptation of Austroads Part 5 see also Austroads technical basis (Austroads 2008)	AASHTO (1993)	Austroads General Mechanistic Procedure (Austroads 2011)

The range of uses of the outputs of this task include:

1. computing a pavement structural health indicator for screening potential candidate treatment lengths, and for network reporting purposes, this being the main purpose of this project task
2. informing options in a Life Cycle Cost (LCC) analysis, including the timing of possible structural treatments
3. providing guidance on the direct use of deflection data, including:
   (a) as an input to network level deterioration models, e.g. to estimate a modified structural number (SNC) to inform the prediction of functional condition
   (b) for identifying potential 'hotspots', which may require repairs.

For this task, a guiding principle has been that having defined a measure of pavement life/health this does not mean a treatment is needed, as this will primarily be driven by functional condition. However, a structural view can be used to help select which treatment is appropriate, or not.

3.3 Adopted methods for further detailed review under this project

Two main methods have been selected with the aim of testing their suitability for determining and estimating a pavement structural index. Further details on each method, and on specific options for each, are described below:

3.3.1 Austroads SNC ratio (Incremental pavement deterioration model) – Method A

Except for the AASHTO method, the other methods assume that future pavement life can be estimated from current deflections without considering the current pavement condition (e.g. cracking, rutting, roughness) other than its response to load. In other words, it is implicitly assumed that any deterioration is reflected in the deflections.

However, residual life can also be estimated by road deterioration models that predict the time/loading to terminal pavement conditions based on the current pavement condition (e.g. cracking, rutting, roughness).

There are many road deterioration models in use. The HDM-4 road deterioration (RD) models and similar models of the empirical-mechanistic, deterministic form, such as the Austroads network level models (Austroads 2010a) are of interest since they have been calibrated or developed specifically for Australian conditions. The parameters included in the models allow a wide range of operating and design conditions to be modelled, i.e. they have a quality of being transferrable for application in different locations following calibration.

This means they can calculate the time and traffic loading to terminal conditions defined by different types and levels of deterioration namely:

- percentage area of all and wide cracking
- average rut depth
- roughness.

Importantly, the remaining life calculated using such RD models not only utilises information about the structural adequacy from measured deflections, but also the current condition in terms of cracking, rutting, roughness, etc.

The Austroads network level functional deterioration models for cracking, rutting and roughness (2010a) work in conjunction with a structural deterioration model (Martin & Choummanivong 2018) The models are specific examples of the HDM-style models developed for Australia, i.e. equation forms, including parameters and coefficients have been selected based on statistical analysis using time-series observations from long-term pavement performance (LTPP) studies funded by Austroads and network studies funded by Austroads and various road agencies. Works effects models have also been developed which quantify the improved condition which results from specific treatments (Austroads 2008).

An advantage of the models is they can be employed within Deighton's Total Infrastructure Management System (dTIMS) which is used by MRWA. The latest (and most comprehensive) dTIMS setup available incorporates the following choices:

- use of a structural deterioration model (based on SNC) to inform functional deterioration – this represents a minimum recommended configuration
- use of structural condition (SNC ratio) as a trigger in combination or separate to purely functional triggers.

Both of the above setups have been tested in analysing a VicRoads 23–year PPP with a 35–year analysis period applied within dTIMS to a network comprising over 8000 100 m long road segments.

A specific technical advantage of such models is they model the interaction of different parameters and account for the significant changes in pavement performance which result through the passage of time and traffic, including beyond the so-called 'stitch in time' beyond which there is a risk of accelerated deterioration. Such conditions may exist on the network, or be possible under budget constraint, in which case an incremental model can inform appropriate modelling and the selection of a treatment strategy that minimises costs, either to the road agency or to the economy in terms of (economic) total transport costs.

The Austroads structural deterioration model (Martin & Choummanivong 2018) is based on the Austroads LTPP study which has been undertaken since 1994 with more sites added from 1998 (Martin & Choummanivong 2016). The model is expressed as the ratio of structural number measured against the initial SNC at the time of construction, and recognises the following influencing factors:

- Pavement type by having a difference in the model coefficients for asphalt and sealed unbound granular pavements
- Pavement age – the pavement will deteriorate over time, in this case with respect to the expected pavement service life
- Effect of climate and environmental conditions which is expressed in terms of the Thornthwaite Moisture Index (TMI). A negative value of the index represents a dry climate and a positive value, a wet climate.
- The initial strength of the pavement at the time of construction is defined in terms of the initial modified structural number, SNC_0. The respective maximum deflections can be used to estimate SNC_0 based on the current modified structural number SNC_i, measured at time 'i':

$$SNC_i/SNC_0 = 2 - EXP[AGE_i*(0.00001942*TMI_i + 0.2975*/DL)] \text{ (for asphalt pavements)} \quad (2)$$

$$SNC_i / SNC_0 = 2 - EXP[AGE_i*(0.00004413*TMI_i + 0.2581*/DL)] \text{ (for granular pavements)} \quad (3)$$

$$SNC_0 = SNC_i/\{2 - EXP[AGE_i*(0.00001942*TMI_i + 0.2975*/DL)]\} \text{ (for asphalt pavements)} \quad (4)$$

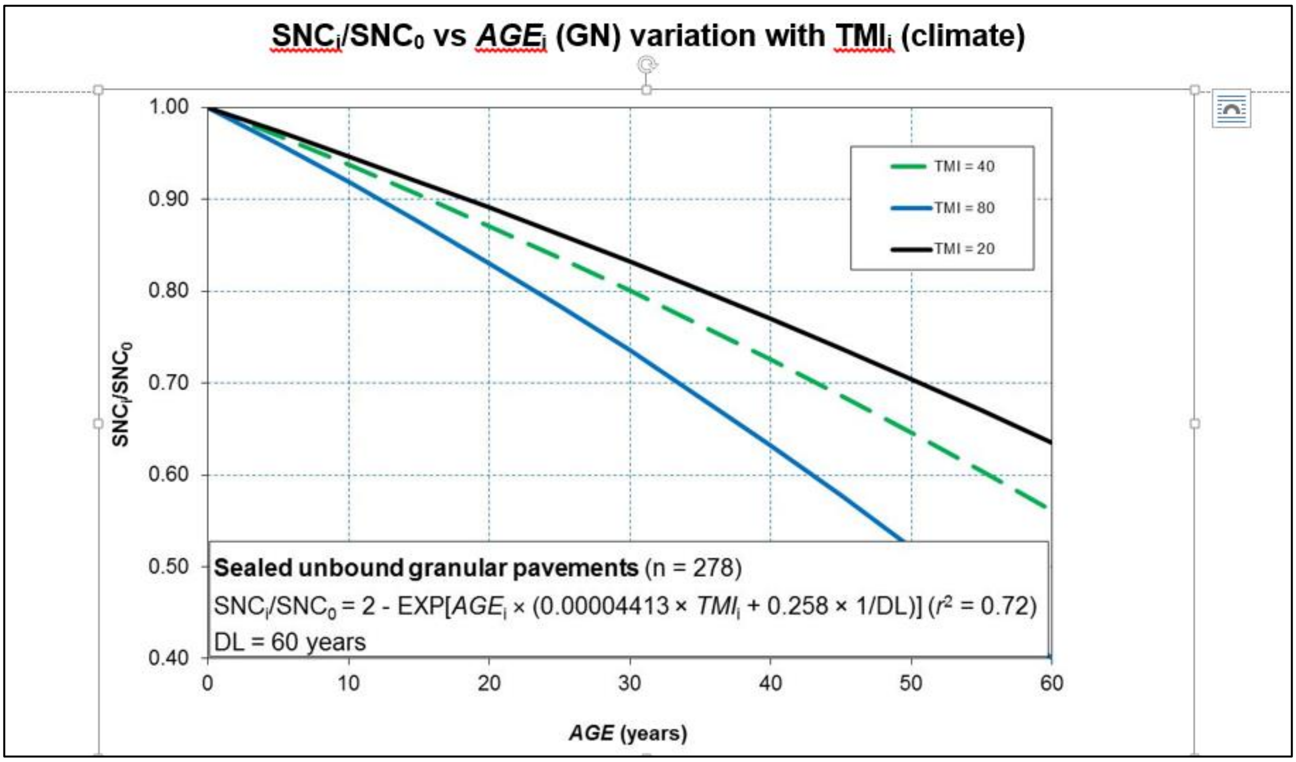
$$SNC_0 = SNC_i/\{2 - EXP[AGE_i*(0.00004413*TMI_i + 0.2581*/DL)]\} \text{ (for granular pavements)} \quad (5)$$

where

SNC_0	=	initial SNC after construction
SNCi	=	current (in-service) SNC estimated from measured deflection
AGEi	=	pavement age (number of years since construction or last rehabilitation)
DL	=	service life in years
TMI	=	the Thornthwaite Moisture Index

The progression of the model over time is illustrated in Figure 3.1.

Figure 3.1: Change in SNC with time for sealed granular pavements



The current strength of the pavement is defined in terms of the SNC at the time of the most recent structural testing (time 'i'), calculated based on the measured peak deflection (D_0) under loading. It involved calculating SNCi firstly by converting TSD outputs to FWD values as detailed in Section 2.2, and then converting these to equivalent Benkelman beam ($D_{0\text{-}BB}$) values, and finally applying Equation 6 and Equation 7 from Patterson (1987).

$$SNC_i = 3.2 * D_{0\text{-}BB}^{-0.63} \text{ (for unbound pavement) or} \quad (6)$$

$$SNC_i = 2.2 * D_{0\text{-}BB}^{-0.63} \text{ (for bound and asphalt pavement)} \quad (7)$$

Note that in using SNC ratio either as input to calculate the remaining life or as a trigger input in dTIMS, if the SNC_0 is back-calculated, the SNC ratio is not dependent on deflection nor curvature but on climate, pavement age and service life only based on Equation 4 and Equation 5.

As an alternative to the back-calculated SNC_0, the initial structural number can be estimated as expressed in Equation 8 (Martin & Choummanivong 2018, Toole & Roper 2014) with knowledge of the estimated design traffic in cumulative equivalent standard axles (CESA).

$$SNC_{0\text{-}est} = 1.128 \text{ CESA}^{0.1033} \quad (8)$$

For the purpose of comparing the adopted methods, both SNC ratio with back-calculated SNC_0, annotated as SNCratio_bc and SNC ratio with estimated SNC_0, annotated SNCratio_est were employed.

3.3.2 Empirical deflection-life based methods

Methods available include the TRRL deflection-life curves which was one of the first methods of its kind and is used to illustrate a well-established method applied over many years, and two examples which are available in Australia and could be applied to estimate a remaining pavement life, albeit with specific qualifications, including those described below.

TRL deflection-life curves

For over 40 years deflection-life curves have been used in the United Kingdom to estimate residual life, with specific curves developed by TRRL (now TRL) (Kennedy and Lister 1978) based on long-term monitoring of full-scale road experiments. These have been revised several times as more data have become available. Deflection-life curves have been developed for the following pavement types:

- pavements with granular road bases whose aggregates exhibit a natural cementing action
- pavements with non-cementing granular road bases
- pavements with bituminous road bases
- pavements with cement bound road bases.

Pavement deterioration on all road experiments was rutting in the wheel-paths, measured under a 1.8 m straight edge. According to Kennedy and Lister (1978):

> Cracking of the road surface only occurs at a relatively late stage as deformation continues until, at failure, the road is badly deformed and may also be badly cracked. Critical condition, defining the preferred time for extending the structural life of a pavement by overlaying is normally characterised by moderate rutting with little or no cracking.

Critical pavement condition was defined as:

- No cracking, rutting 10 mm to 19 mm; or
- Cracking confined to a single crack or extending over less than half of the width of the wheel path. Rutting 19 mm or less.

These modes of distress are consistent with the predominant distress modes such as vertical strain and asphalt fatigue applied in empirical, deflection-based methods. Advanced deterioration is reflected in increased rutting, surface distress and roughness.

Figure 3.2 is an example of the relationship between the TRRL standard deflection (Benkelman beam deflection under a wheel load of 3175 kg, corrected to a temperature of 20 °C) and the pavement life to a critical condition for pavements with granular road bases whose aggregates exhibit a natural cementing action. Such deflection-life curves enable the residual life (RL) to be calculated from the measured deflection and cumulative traffic loading at the time the deflections were measured. For example, as seen from Figure 3.2 if the measured deflection was 0.45 mm and the cumulative standard axle loading was 3.0×10^6 ESA (point B in Figure 3.2), the allowable loading to critical condition at a probability of 0.5 is 6.0×10^6 ESA. Consequently, the residual life percentage is:

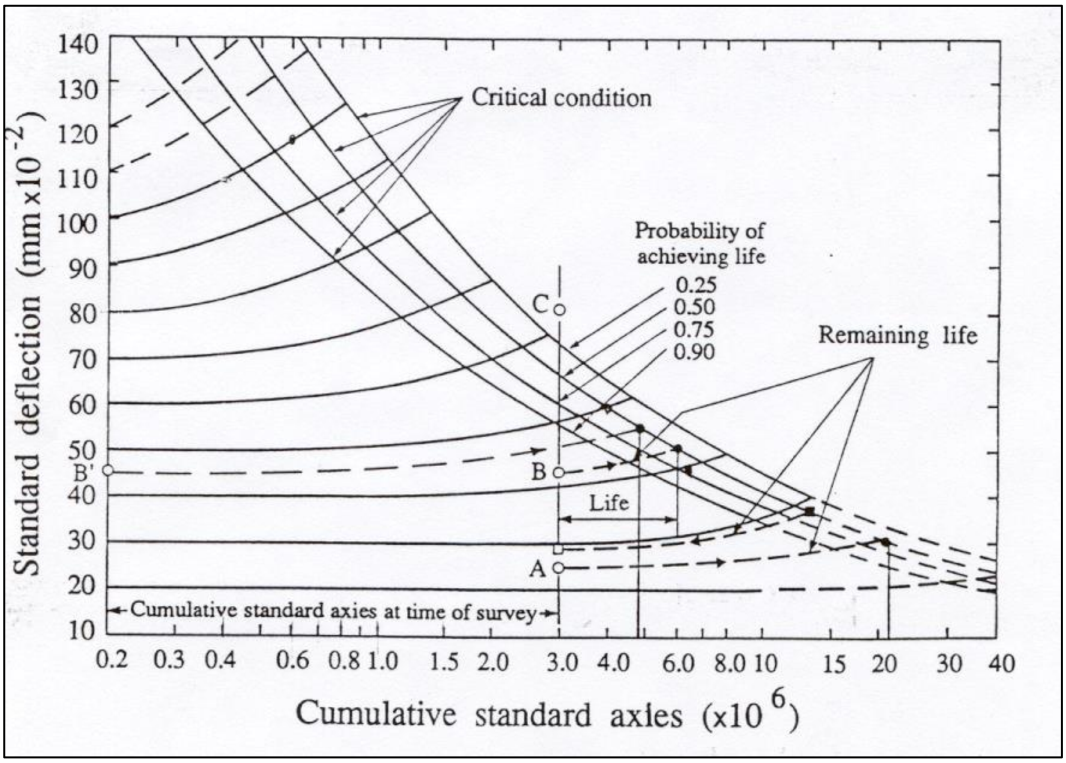
$$RL = 1 - \frac{3.0 \times 10^6}{6 \times 10^6} = 50\% \qquad (9)$$

where

RL = percentage residual life (%)

or alternatively, the pavement can withstand an additional loading of 3.0×10^6 ESA.

Figure 3.2: Relationship between standard deflection and life for pavements with granular road bases whose aggregates exhibit a natural cementing action – design example



Source: Kennedy and Lister (1978).

Adaptation of the Austroads pavement rehabilitation method (Method B)

An adaptation of the Austroads pavement rehabilitation method has been developed which involves determining the allowable loading of nominal treatments on an existing pavement. This can be converted into a 'Notional structural life' (NSL) and banded into years as a structural demand index (SDI). The method has also been applied in VicRoads PPP development to define an initial distribution of NSL and potentially as a monitoring tool to complement functional condition profiles. The application is similar to the former MRWA Term Network Contract (TNC) Asset Condition Profiles[2].

The method involves determining the distribution of estimated lives (in years) of nominal resurfacing treatments applied to individual 100 m sections of a network, as follows:

- For thin bituminous surfaced granular pavements with either a bituminous seal or asphalt less than 40 mm thick the treatment is reshaping and resealing without strengthening. The allowable loading is based on a relationship between the characteristic (design) deflection and design traffic loading taken from Austroads (2011) *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* (Figure 3.3). In this procedure the before and after deflection remains the same, i.e. there is no net strengthening.

[2] The term 'notional structural life' does not imply a definite structural life. Instead it is based on the concept of a structural treatment demand, which in this case is represented by an allowable loading converted into years. The concept is useful as a health index and the value can be redetermined periodically using a standard procedure, i.e. it can be monitored. It also provides a basis for comparison with methods which aim to estimate remaining/residual life. A further use is to inform possible treatment options, and their timing.

- For asphalt pavements, the calculated lives are those after resurfacing with a 50 mm thick asphalt overlay. The allowable loading is based on a relationship between the characteristic (design) curvature and allowable traffic loading adapted from Austroads (2008) *Technical basis of the Austroads design procedures for flexible overlays on flexible pavements, Research report AP –T99/08* (Figure 3.4). In this procedure the before and after deflection differs, i.e. net strengthening results from the nominal treatment.

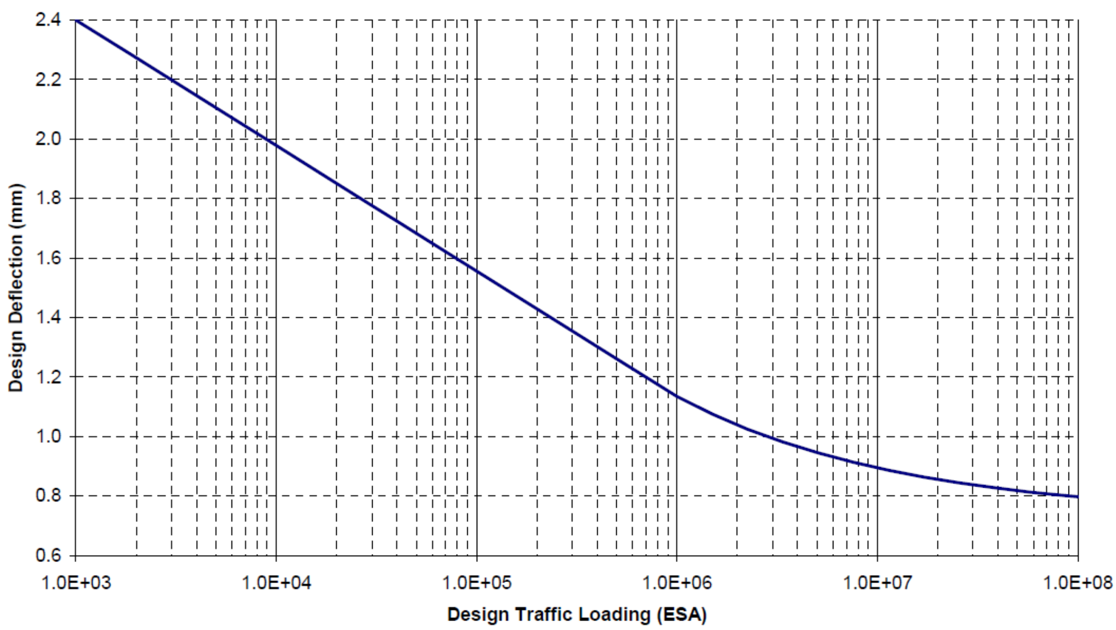
Figure 3.3: Benkelman Beam design deflections

Design traffic less than 10^6 ESA:

$$D = 3.666 - 0.422 \log N_{ESA} \qquad 1$$

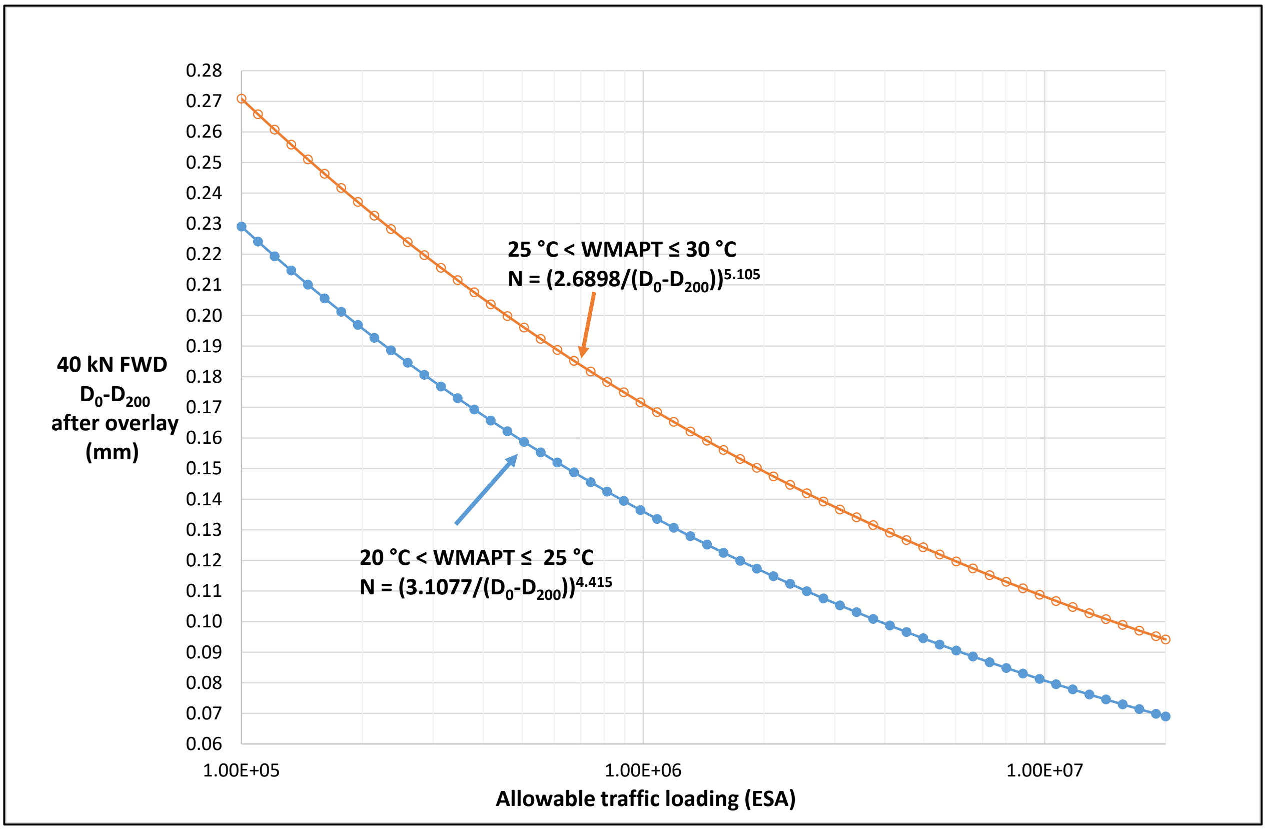
Design traffic greater than 10^6 ESA:

$$D = 0.731 + 91.202 N_{ESA}^{-0.3924} \qquad 2$$



Source: Austroads (2011).

Figure 3.4: 40 kN FWD design curvatures for 50 mm thick asphalt overlay for WMAPTs of 20–30 °C



Source: Austroads (2008).

In a recent application in Victoria (Toole & Jameson 2017), the above procedures were applied to determine pavement performance criteria comprising an indicator of structural treatment demand which would help inform the asset owner of the sustainability of the pavement assets, and the likely challenges to be faced by the project contractor in managing the network. The need for such an indicator is based upon concerns that traditional condition indicators could be achieved through the frequent placement of relatively light treatments, i.e. band-aiding, and that the 'asset owner' and, indeed, the contractor could be at risk from the accelerated deterioration of the road pavements, either during or immediately after the duration of the concession.

For network monitoring purposes, the results of the analysis are presented as a cumulative frequency distribution, similar in form to the asset condition profiles (ACP) employed by MRWA. The option is also to use the data for individual segments to inform possible treatment options, in particular where there is evidence of distress.

3.3.3 The ARRB/RMS structural evaluation of pavements (STEP) procedure – Method C

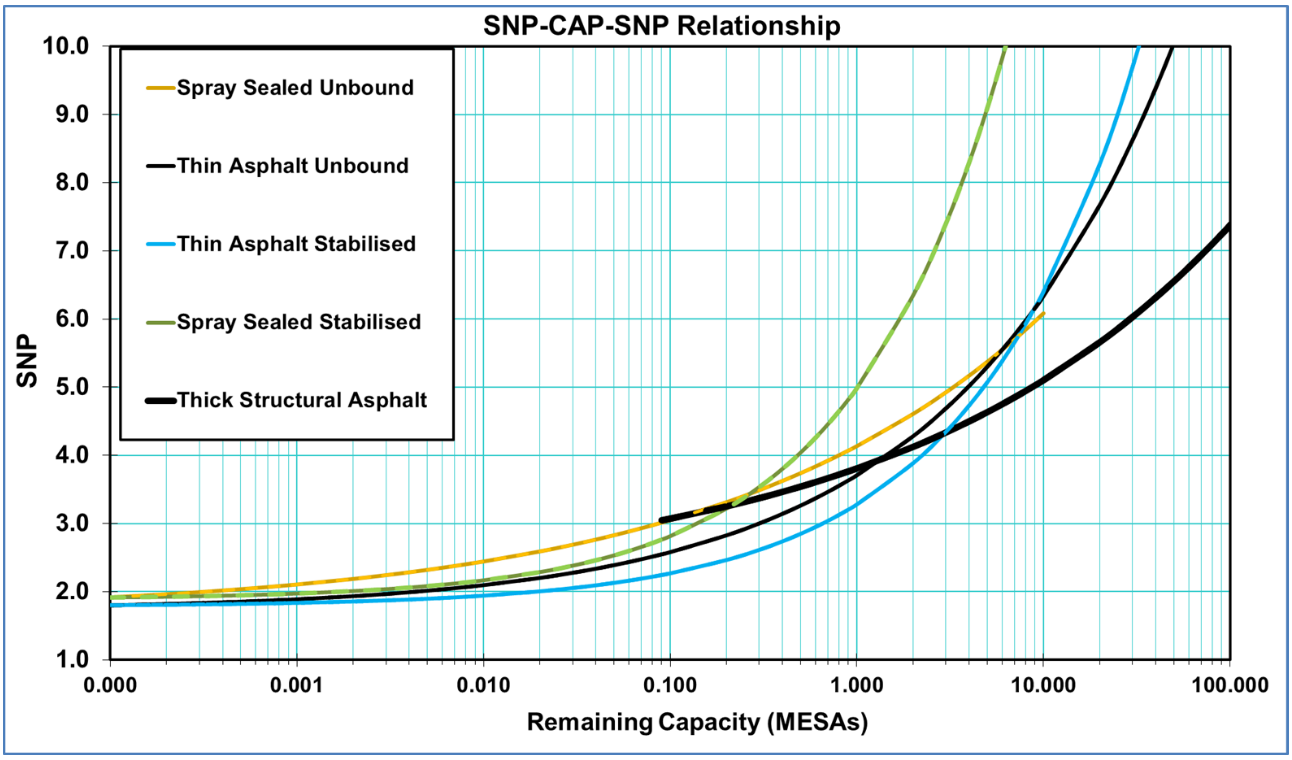
The STEP procedure has been applied by RMS to augment their functional modelling by providing an estimate of remaining structural life. At its core is the following two calculations:

- Determination of the current SNC of a pavement accounting for multiple factors, including:
  - layers; surface, base, subbase and subgrade
  - material type per layer and overall pavement type
  - thickness of each layer

- estimated material strength of each layer accounting for temperature and moisture conditions, and material integrity

- Calculation of the remaining structural life (RSL) in ESA based on the current SNC, with the relationship established from mechanistic pavement analysis and validated by asset management practitioners (Figure 3.5).
- Adjustment of the RSL to account for the stiffness of the pavement as an indicator of the performance of the upper layers of the structure, noting this relates to structural crack initiation in asphaltic layers. The indicator used is the curvature, with the concept applied to the stiffness of any flexible pavement structure, including cemented and unbound sprayed seal pavements, with either strong or weak road bases. This contributes to the derivation of a term named the Pavement Stiffness Ratio (PSR) which is used to adjust the RSL for cases where the PSR is less than unity.

Figure 3.5: Example pavement in-service remaining capacity curves



Source: Roberts (2017).

Data for the adapted Austroads deflection-based procedure is readily available, whereas a simplified approach has been adopted for the STEP method considering the following:

- estimation of RSL based on maximum deflection (D_0) only, following conversion to SNC
- adjustment of RSL accounting for PSR.

In coming to the above set of options there is a need to focus and test potential methods, including alternative procedures, and not broaden the study too widely bearing in mind this is a network level study and needs to draw as much as possible on Australian methods.

3.4 Structural index comparison

3.4.1 *Description of the selected setups*

This section compares the three selected structural index methods discussed previously in Section 3.3 by expressing the calculated values as the remaining structural life (RSL). The results have been plotted using a visualisation tool developed by the project in the Microsoft Power BI platform. The functional conditions have also been plotted to help illustrate their relationship to the estimated structural capacity.

Table 3.2 provides a summary of the scope and input data prepared for the RSL calculation for each method. The choice of a 60–year design life (with the latter term replaced by the term service life which is consistent with the derivation of the models) is based on the fact that roads last a long time, particularly in WA, without a need for a rehabilitation or other major strengthening treatment.

Table 3.2: Input data requirements

Input data	Method A – Austroads SNC ratio	Method B – Notional structural life	Method C – ARRB STEP procedure
Data interval	100 m	100 m	100 m
Deflection	TSD to FWD conversion average aggregation	TSD to FWD conversion average aggregation	TSD to FWD conversion average aggregation
Curvature	Not used	TSD to FWD conversion average aggregation (for asphalt only)	TSD to FWD conversion average aggregation
Design life	60 years (service life)	60 years (service life)	Not required
Pavement type	2 types of surfacings: – Asphalt – Sprayed seal	2 types of pavement: – Asphalt (similar to STEP P5) – Granular	5 types of pavement: – P1 Sprayed seal unbound – P2 Sprayed seal stabilised – P3 Asphalt unbound – P4 Asphalt stabilised – P5 Full depth pavement
Calibration	Austroads LTPP	Not required	ROCe (Stiffness coefficient) derived from TSD 900 dataset for each pavement type sub-group)

In summary the methods are based on the following:

- Method A considers the ratio of structural number when deflection was measured against the structural number immediately after construction. It assumes that the structural capacity will decrease over time until the ratio reaches a certain point when the structural life is deemed to have been consumed. For this study the SNC ratio value of 0.59 was adopted. Two alternative ways of calculating the RSL were considered depending on how the SNC_0 was calculated. The first alternative is for when SNC_0 was back-calculated (RSL_SNCratio_bc) and the second for when SNC_0 was estimated with an empirical equation (RSL_SNCratio_est).

- Method B, the Austroads notional structural life method (RSL_NSL), converts the allowable traffic for a nominal treatment into a remaining service life (Toole & Jameson 2017).
- Method C, the ARRB STEP method, estimates the remaining life (RSL_STEP) based only on the structural integrity of the pavement while accounting for a potentially weaker upper layer of the pavement likely due to cracking.

The maximum reported RSL for all three methods was limited to be no more than 80 years.

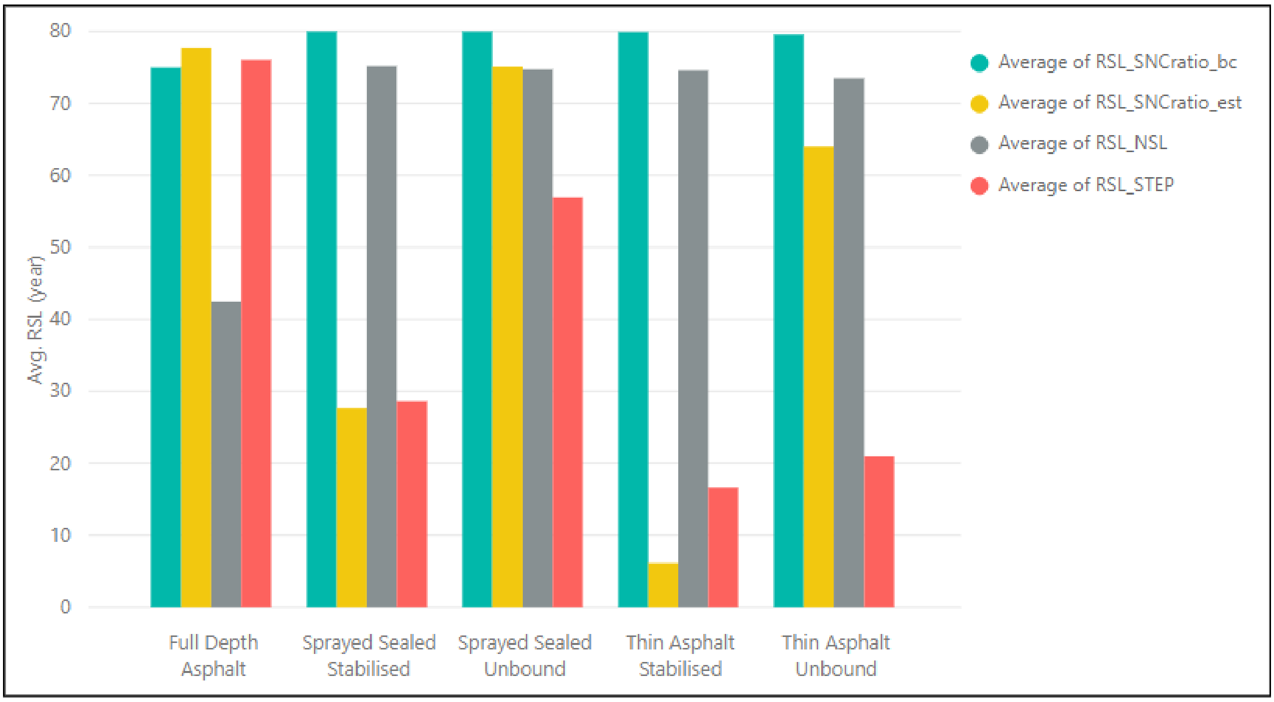
3.4.2 *Results*

Average remaining service life by pavement type

Figure 3.6 presents the calculated average remaining service life by pavement type sub-group estimated from applying each of the methods, from which the following observations were drawn:

- Method A, RSL_SNCratio_bc, gives a constantly high RSL regardless of the pavement type. The result of the alternative application of Method A, using RSL_SNCratio_est, however varies considerably by pavement type, and is especially low for thin asphalt on a stabilised pavement. The difference between the two alternatives is significant when the base is stabilised.
- Method B, RSL_NSL, gives a relatively high RSL except for the case of a full depth asphalt pavement where the estimate is the lowest of all three methods.
- Method C, RSL_STEP, follows a similar trend to Method A based on an estimated SNC_0, RSL_SNCratio_est. An exception is when the base is an unbound pavement where Method C tends to predict a much lower remaining structural capacity.

Figure 3.6: Comparison of the remaining service life



In a further example, representing the Kwinana Freeway (H015) which is a typical highly-trafficked major road in an outer urban setting, the effect of the variation in pavement type has been investigated in more detail utilising the Power BI visualisation tool as shown in Figure 3.7. From the top, the first chart provides a comparison of methods of estimating RSL, the second chart is a combination of curvature readings from the TSD survey and pavement type, and the third chart presents rutting and cracking profiles and the last chart presents roughness and cracking profiles. A constant line to represent a trigger level is provided for each condition parameter; 300 for curvature, 20 mm for rutting and 4.2 IRI for roughness.

The following observations can be made:

- Method A, RSL_SNCratio_bc shows a constantly high RSL estimate. Similarly, Method B, the RSL_NSL method, also produces a high estimate except for the full-depth asphalt section.
- The fluctuations shown for Method A, RSL_SNCratio_est and Method C follow that of the measured pavement stiffness, represented by D_0. When the pavement is very stiff it produces a relatively high remaining life estimate as observed between SLK 14 to 22 for the thin-asphalt unbound pavement, and from SLK 41.5 to 57.0 which comprises full-depth asphalt.
- There is no significant difference between the Method C RSL calculated for the thin-asphalt surfacing with a bound or unbound base configuration as observed from SLK 22 to SLK 42. However, for Method A RSL_SNCratio_est, there is a significant difference in the calculated RSL between the two pavement types, with the thin-asphalt on stabilised pavement having a much shorter estimated life for a similar curvature or deflection.
- For sprayed seals, a noticeable difference of results between bound and unbound pavements was observed for Method C. The sprayed seal-unbound section (SLK 340 to SLK 348) has a higher RSL estimate, approximately twice as high, than the sprayed seal-stabilised section (SLK 348 to SLK 354) for a comparable measured curvature. This is however consistent with the different relationships shown in Figure 3.5.
- When a pavement is in good condition, with no indication of surface defects, and it possesses low deflection and low curvature, the RSL estimate varies by pavement type quite considerably. Method C predicts the lowest RSL, in the 30s, Method A estimates RSLs in the 80s and Method B in the 60s.
- On the other hand, when pavements are in poor to very poor condition, where defects can be visually observed, Method B and C are consistent in estimating low RSL as shown in Figure 3.8 from SLK 355 onwards. The visual evidence is provided in Figure 3.9 for SLK 372 on the Great Eastern Highway. The image shows a high frequency of maintenance patching, crocodile cracking and rutting on the wheel path. For this example, the estimated RSL from Method A remains high. This is because the SNC ratio used in determining the remaining structural life for Method A, when using back calculation to estimate SNC_0, is dependent only on the climatic condition, pavement age and the service life as shown in Equation 2 and Equation 3. However, it is understood that this method needs to be used in conjunction with functional data.
- Method A with the back-calculated SNC_0 and Method B produced a constantly high value of RSL with the former showing little response to fluctuations in measured deflections.

Figure 3.7: Kwinana Freeway (H015) Structural Capacity Comparison

Figure 3.8: Great Eastern Highway (H005) Structural Capacity Comparison

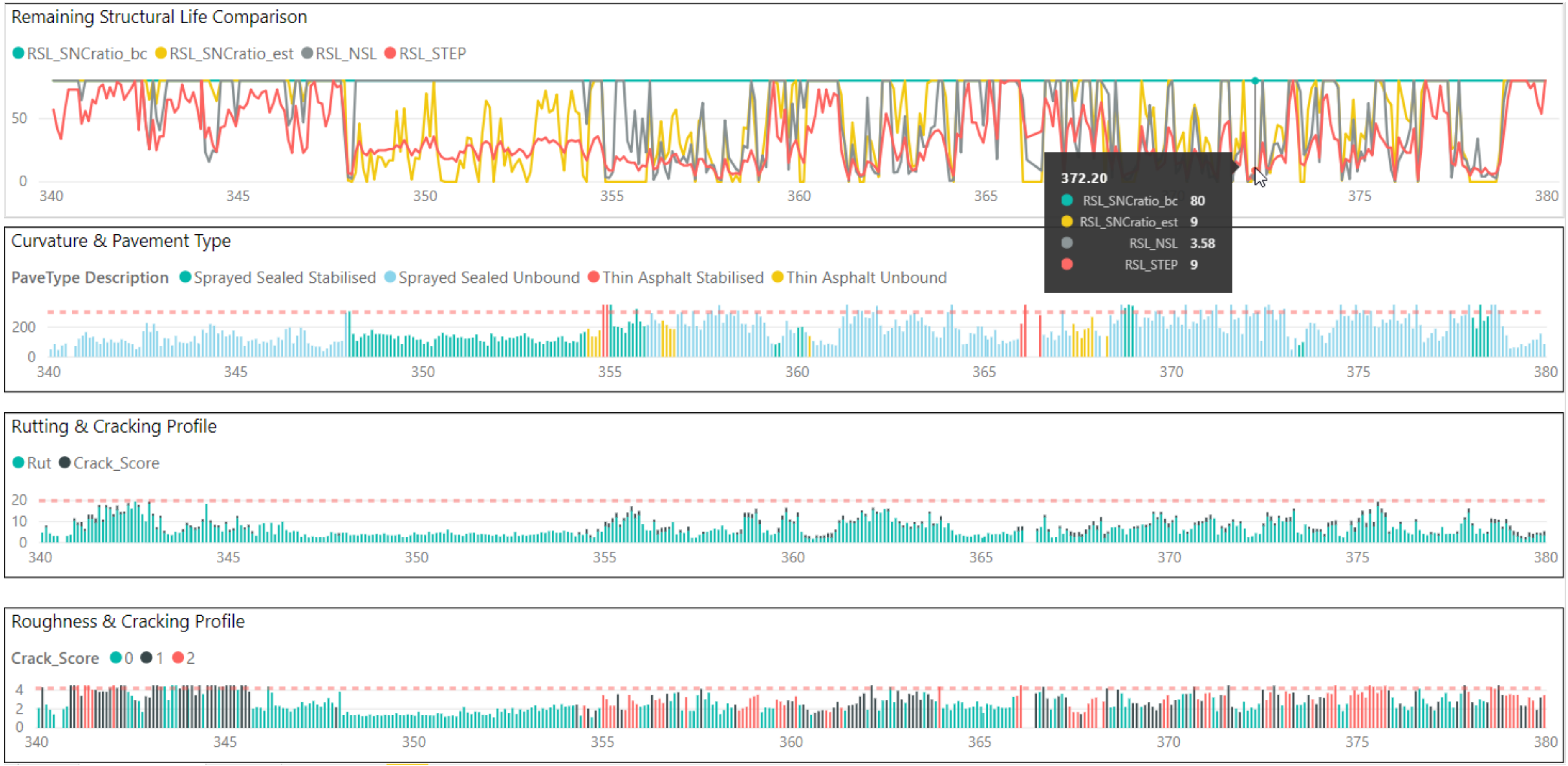


Figure 3.9: Great Eastern Highway visual condition at SLK 372

4 REVIEW OF FACTORS INFLUENCING STRUCTURAL DETERIORATION (TASK 1B – 1C)

4.1 Statistical analysis approach

The following methodology was used in determining factors that might influence pavement structural deterioration in Western Australia.

- Determine sample size. There are over 14,400 individual 100 m deflection readings but for this exercise, just under 1,800 samples were used. They have been pre-screened for the accompanied poor functional condition, i.e. for high roughness or rutting or cracking exceeding treatment triggers as per the MRWA pavement modelling specification (MRWA 2018).
- Determine the factors (independent variables) that might influence the dependent variables.
- Determine the dependent variables to test the independent variables.
- Undertake statistical analysis to investigate a correlation between each independent variable and the dependent variable

4.2 Factors influencing structural deterioration

The project scope sought to identify the environmental and geological factors and other factors or parameters that might influence pavement structural integrity. Those investigated however are limited to the available data.

For the general environmental factor, the Thornthwaite Moisture Index (TMI) is used to define different climate zones. This is the same climatic factor used in the Austroads Structural model as discussed in relation to Method A (Section 3.3.1). Other factors included in the analysis were temperature and precipitation data sourced from the Australian Bureau of Meteorology (BOM).

The geological factor is represented by the Western Australia soil classification data provided to ARRB by MRWA.

4.2.1 TMI calculation

TMI was calculated using the method in Austroads (2010c). TMI is a combination of the annual measures of precipitation, evapotranspiration, soil water storage, moisture deficit and runoff (Thornthwaite 1948). It uses historical precipitation and change in temperature data from BOM in the last 10 years from the relevant weather stations. A negative value of TMI represents a dry (moisture deficit) area and a positive TMI represents a wetter (moisture surplus) climate.

The list of the stations, their coverage and the calculated TMI are shown in Table 4.1 with complementary information on mean-monthly temperature and total monthly precipitation.

Table 4.1: TMI coverage of the TSD 900

Station	RoadID	SLK_start	SLK_end	Temperature (Mean monthly)	Precipitation (Monthly total)	TMI
Madura Station	H003	525	595	23.7	30.3	–49.69
Mundrabilla	H003	595	675	23.7	30.3	–50.14
Eucla	H003	675	750	23.7	30.3	–49.07

Station	RoadID	SLK_start	SLK_end	Temperature (Mean monthly)	Precipitation (Monthly total)	TMI
Perth Airport	H005	0	90	25.5	57.0	–40.27
Northam	H005	90	125	26.2	37.0	–47.53
Cunderdin	H005	125	175	27.0	18.4	–54.61
Kellerberrin	H005	175	225	26.2	27.9	–57.37
Merredin	H005	225	325	26.0	26.2	–56.64
Southern Cross Airfield	H005	325	475	26.1	22.0	–56.14
Coolgardie	H005	475	525	26.1	22.0	–47.58
Fremantle	H015	0	75	25.5	57.0	–48.31
Perth Regional Office	H018	0	50	25.5	57.0	–38.03
Perth Regional Office	H021	0	25	25.5	57.0	–38.03
Perth Regional Office	H057	0	40	25.5	57.0	–38.03

Source: Australian Bureau of Meteorology (BOM) accessed June 2018

The area where the TSD survey was undertaken lies roughly along the same latitude as Perth going west to east as shown in Figure 2.1. The climatic zone for the area was found to be similar, and ranges from a TMI of –38 in and around Perth to –57 around Eucla, both are dry to very dry climate zones.

4.2.2 Soil classification

ARRB was provided by MRWA with a set of soil classification data that reflects the change in the underlying soil by sections marked by start and end SLK. The data was not as readily usable as the climate data as there were numerous overlaps between the sections with no clear single classification. Consequently, the data was manually reclassified into the classifications shown in Appendix C. This uses general groupings of sand, gravel, loam, and clay soils.

4.2.3 Dependent variable

Once the factors to be investigated were determined, the question of what continuous variable should be the dependent variable remained.

Ideally, a way to measure changes in the structural behaviour of a pavement should be adopted as the dependent variable such as the time-series deflection data. In the absence of such data, initially a maximum deflection was considered since it has also been used as a treatment trigger in MRWA’s pavement management system with a threshold of 0.9 mm applied (MRWA 2018). However, in the opinion of the project team, this should be used in conjunction with the structural demand expected by the pavement, as high deflections on certain roads might be more tolerable in the case of lightly trafficked roads than for higher traffic. It was therefore decided that for the purpose of investigating a correlation between dependent and independent variables, the remaining structural life for ARRB STEP (Method C) would be used. This method was chosen because it shows the highest range of RSL estimates, with this potentially being associated with environment and geological factors.

A summary of the factors considered under this study is shown in Table 4.2.

Table 4.2: Summary of factors influencing structural deterioration considered

Factors	Column code	Type	Used in stats. analysis
Roughness	IRI	Numerical	Independent variable
Rutting	Rut	Numerical	Independent variable
Cracking	Crack	Numerical	Independent variable
Pavement age	Pavement age	Numerical	Independent variable
Climate	Temperature (Mean monthly)	Numerical	Independent variable
Climate	Precipitation (Monthly total)	Numerical	Independent variable
Climate	TMI	Numerical	Independent variable
Geological	Soil classification	Categorical	Independent variable
Remaining life	RSL_STEP	Numerical	Dependent variable

4.2.4 Influence of environmental factors

A correlation analysis was conducted with the summary results provided in Table 4.3. A significant influence of any single factor is shown by a high negative correlation. The last row shows the Pearson correlation values between the dependent variable, RSL_STEP, and the independent variables.

Table 4.3: Correlation result for environmental factors

Variable	IRI	Rut	Cracking	Pavement age	Temperature	Precipitation	TMI	RSL_STEP
IRI	1							
Rut	0.02	1						
Crack	0.00	–0.12	1					
Pavement age	0.01	–0.11	0.03	1				
Temperature	0.33	–0.18	0.10	0.07	1			
Precipitation	–0.36	–0.14	–0.08	–0.09	–0.43	1		
TMI	–0.18	0.04	–0.30	–0.31	–0.36	0.64	1	
RSL_STEP	0.13	–0.05	0.02	–0.10	0.1	–0.24	–0.09	1

No significant correlation was observed between the dependent variables with temperature or precipitation or TMI. It appears that precipitation has a relatively stronger correlation, albeit a weak one, compared to TMI. It should be noted that the variations of TMI and temperature are very small for this data set. A similar exercise with inclusion of roads in much wetter climates and with more variation in temperature is recommended to confirm the above observations.

It should also be noted that a very weak negative correlation corresponding to pavement age is observed suggesting that structural age has almost no impact on structural deterioration. This might be attributed to the dry climate condition where the survey data was collected. The observation of the LTPP FWD data (Austroads 2010b) indicated that pavement base and subgrade were getting stronger as a result of the long-term drying condition and not always steadily deteriorating.

Little or no correlation is also observed between the functional parameters (roughness, rutting or cracking) and RSL_STEP.

4.2.5 Influence of geological factors

Since the Soil Classification factor involves grouping on the basis of the soil type, a multiple regression of the dependent variable RSL_STEP and the estimated value for each soil type group was performed with the result shown in Figure 4.1. A low R-squared value was observed which means the model can only explain 13% of the calculated RSL_STEP. Based on the available data, the influences of soil classification on structural deterioration are therefore very weak.

Figure 4.1: Multiple regression results based on geological factors

```
Coefficients:
                                          Estimate Std. Error t value Pr(>|t|)    
(Intercept)                                20.6333     3.6581   5.640 1.97e-08 ***
Soil_ClassificationClay_Loam_Sand_Waterlogged  0.3667    20.3677   0.018  0.98564    
Soil_ClassificationGravel                   2.1167     6.2027   0.341  0.73296    
Soil_ClassificationLoam                    25.1043     3.7915   6.621 4.70e-11 ***
Soil_ClassificationLoam_Sand                1.5667     9.6785   0.162  0.87143    
Soil_ClassificationLoam_Sand_Gravel        37.4549     5.0189   7.463 1.32e-13 ***
Soil_ClassificationLoam_Sand_Rocky         21.5974     6.6531   3.246  0.00119 ** 
Soil_ClassificationLoam_Waterlogged        23.4011     5.2178   4.485 7.76e-06 ***
Soil_ClassificationSand                    20.7556     4.9532   4.190 2.92e-05 ***
Soil_ClassificationSand_Gravel             35.9596     3.7062   9.703  < 2e-16 ***
Soil_ClassificationWaterlogged             18.5351     4.1962   4.417 1.06e-05 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

Residual standard error: 20.04 on 1787 degrees of freedom
  (1 observation deleted due to missingness)
Multiple R-squared:  0.1352,    Adjusted R-squared:  0.1304 
F-statistic: 27.95 on 10 and 1787 DF,  p-value: < 2.2e-16
```

More work is needed in classifying the WA soil data to better reflect the soil subgrade, such as soil reactivity, although it is noted that an additional descriptor (non-cracking or cracking) is represented in the full description. As an example, TMR in Queensland employ a combination of soil reactivity and the climate zone (wet or dry) to identify potential impacts on pavements. In Victoria, roads built on reactive soils have been shown to have an almost two-fold difference in the age-related rate of deterioration (Martin, Toole & Oliver 2004), with lighter, low volume pavements displaying much higher absolute rates of increase in roughness than pavements designed and operating under heavy traffic.

5 CASE STUDIES (TASK 1D) – NETWORK LEVEL APPLICATION – LIFE CYCLE COST ANALYSIS WITH DTIMS

5.1 Background

One of the main goals of having a network wide structural strength assessment was to use it in a PMS environment to inform more accurate network wide programming. For this the Deighton's Total Infrastructure Management System (dTIMS), a common PMS platform that MRWA and ARRB shared was used.

dTIMS has been utilised not only in MRWA but also other state road authorities such as in TMR in Queensland, VicRoads in Victoria and the Department of Infrastructure, Planning and Logistics in the Northern Territory. It provides several optimisation options and an open framework for users to customise data input and analysis models. Users can define:

- the interval at which the input data is analysed
- the number of condition parameters that are applied in the analysis, with this dependent on the user specified road deterioration and work effect (RDWE) models
- how these parameters interact with each other in setting the limits and triggers to generate a treatment.

The output is a works program for the specified analysis period, budget and the optimisation option. Reports are available in tabular and graphical format covering performance (year-by-year), works programs and financial data.

MRWA have been using dTIMS as its main pavement modelling tool to provide the baseline for their road preservation program since the early 2000s. It is understood that since 2014, significant developments have occurred fully within the dTIMS environment. For this project ARRB was supplied with the most recent draft documentation of MRWA pavement modelling procedure (MRWA 2018), and MRWA applied this in their analysis.

ARRB's own dTIMS setup has evolved over the years with a typical base model utilising the deterioration models from Austroads. The base setup is then customised to the needs of the end user typically by adjusting the number and cost of treatments and their triggers, with the optimisation option chosen to suit the strategic goal of the client. Where possible, the RD and WE models are calibrated to local conditions. The last iteration of the ARRB setup was developed for VicRoads. It includes the Austroads functional models with the structural model used to inform the rutting and roughness model. In addition, the Austroads structural model (Method A) and/or the Austroads overlay design approach (Method B) can be applied as a limit to trigger rehabilitation. This formed the template for the ARRB setup adopted for this project with the following considerations:

- Data input employed the same 500 m segmentation and data from the MRWA dTIMS analysis with the exception of the deflection and curvature data which were provided by ARRB from the TSD survey.
- Cracking data were based on the 0, 1 and 2 rating from the MRWA visual assessment. These were converted to cracking values (by extent) of 0, 8 and 25% respectively to utilise the Austroads cracking model employed in the ARRB dTIMS setup.

- Two sets of data input were prepared with these based on the aggregation method used, one where the average was used when aggregating from 100 m sections to 500 m, the other based on a 75th percentile aggregation. The project team only used the aggregated 'average' data.
- Treatment trigger rules for the project followed those of MRWA's with some adjustments made to replace the curvature and deflection with either SNC ratio or the Notional Structural Life (NSL) as a 'basic' rule or limit for triggering a major treatment. As an example, the modified treatment trigger table for road link category AW is provided in Appendix B.
- The ARRB's five-point scale Pavement Condition Index (PCI) was converted to match the four-point scale of MRWA's Pavement Health Index (PHI).

5.1.1 Comparison of the model setups and predictions

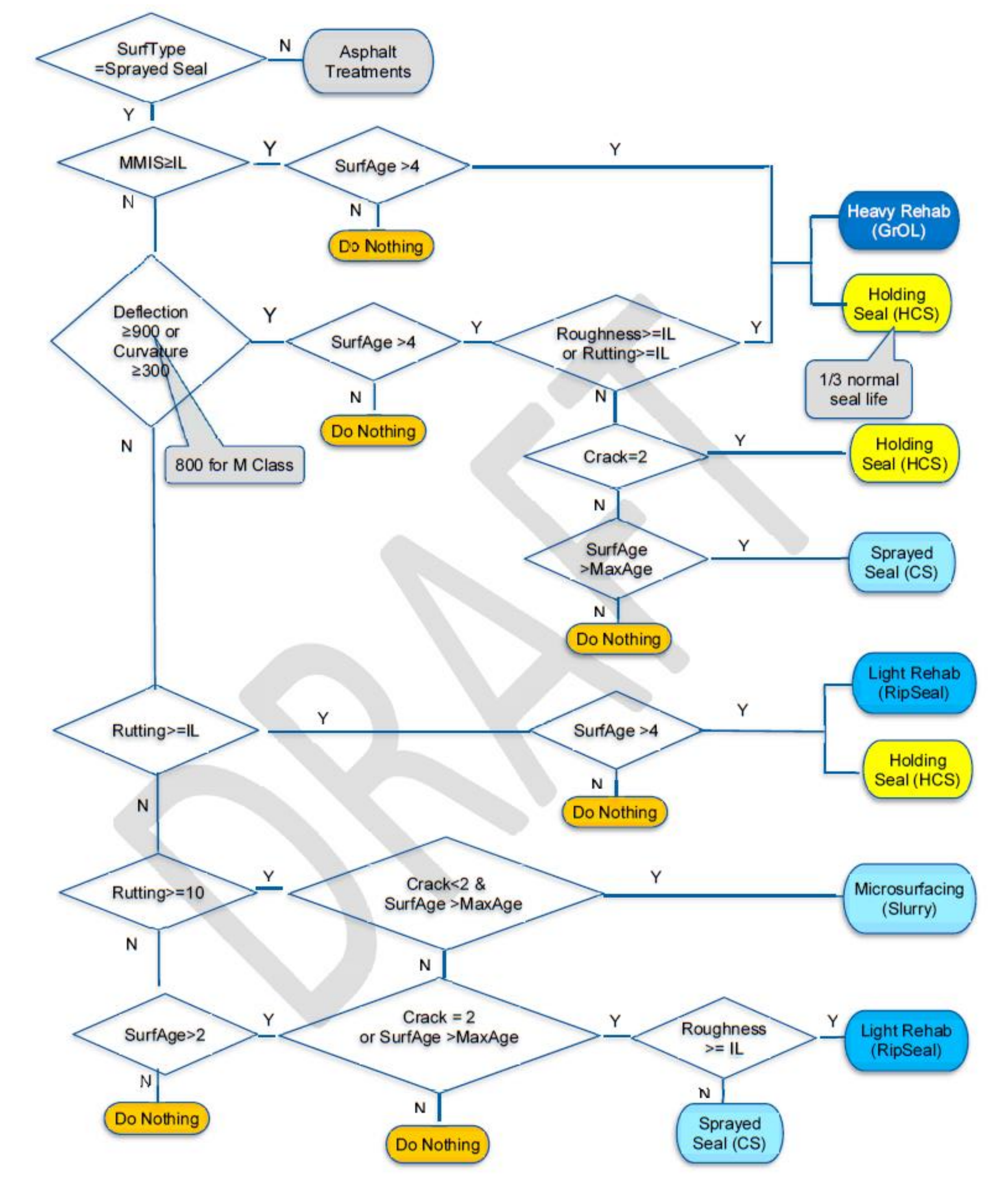
MRWA model

MRWA considers functional as well as structural condition in their pavement modelling procedure. A set of treatment tables for each link category has been developed (MRWA 2018), where treatments are triggered by trigger value(s) where all 'basic' conditions are satisfied based on a comprehensive treatment selection flowchart available for both asphalt surfaced pavements and sprayed seal granular pavements, with the latter presented in Figure 5.1. A number of the main rules are summarised below:

- An MMIS index is employed to identify the need for heavy rehabilitation, i.e. where upkeep costs are high.
- Deflection and curvature are used as an initial screen to trigger either a holding treatment or heavy rehabilitation treatment depending on functional conditions.
- Functional triggers are applied where deflection and curvature limits are not exceeded with the possibility of either do nothing, or surfacing (including micro-surfacing) or light rehabilitation treatment options being selected.

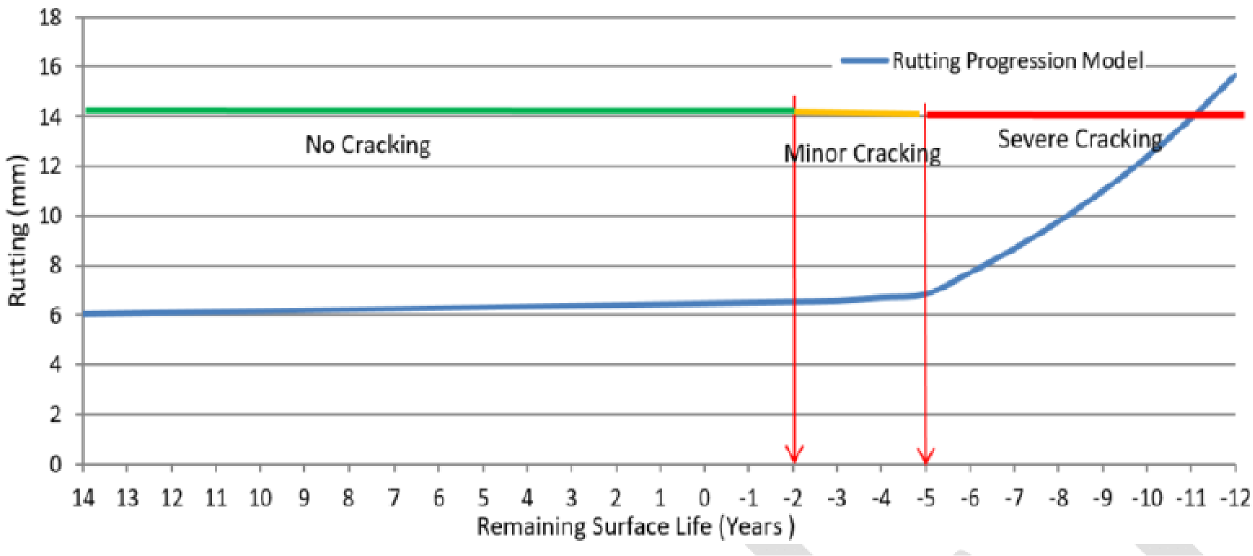
MRWA's modelling procedures acknowledge and employ models representing the gradual and rapid deterioration phases as well as the interaction between functional condition parameters. The impact of traffic and climate on condition progression is also recognised. The procedure relies on the assumption that a good waterproof surface lasts a finite length of time, after which cracking will initiate. The onset of cracking is estimated to occur at 1.1 times the (target) surface life, following which the rapid deterioration phase commences. An example showing the MRWA rutting progression model, as extracted from the modelling procedure, is provided in Figure 5.2.

Figure 5.1: Treatment selection chart for spray sealed granular pavements



Source: MRWA (2018).

Figure 5.2: MRWA rutting progression model



Source: MRWA (2018).

For rutting and roughness progression, MRWA applies an annual deterioration rate during the gradual deterioration phases with a single value used related to road link category. For the rapid deterioration phase, two sets of more aggressive rates are provided with selection based on the severity of cracking, with roads with major cracking assigned considerably higher deterioration rates than those with minor cracking. The latter rates only apply to locations where the annual rainfall is greater than 300 mm.

Different rates of deterioration are also specified for urban and rural roads.

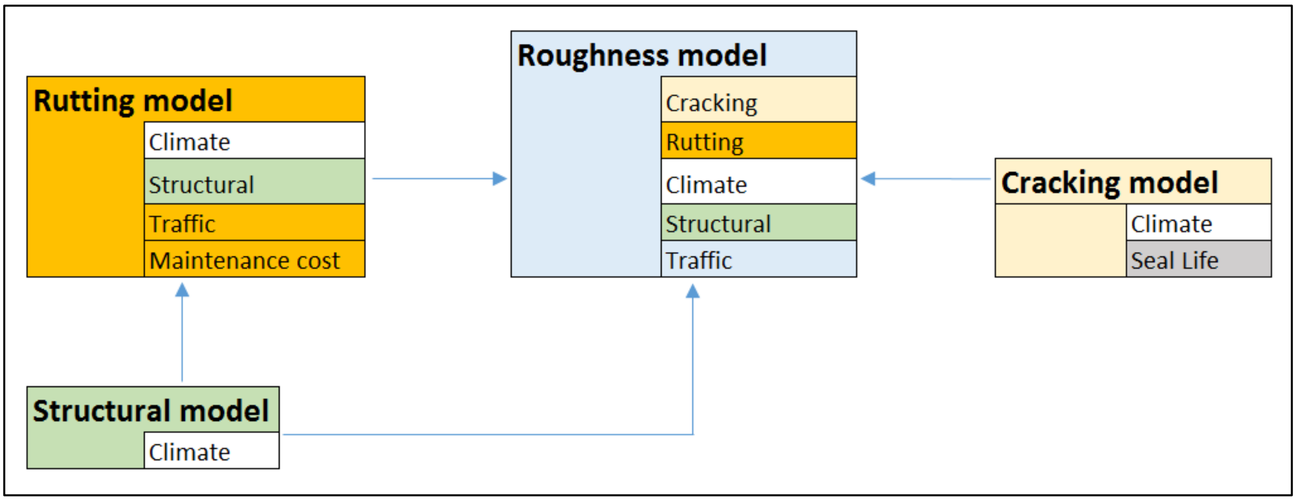
Cracking is based on the annual visual assessment done by MRWA inspectors and hence cracking progression is not modelled.

ARRB model

The ARRB setup employed the Austroads models directly which in their current form only represent the gradual deterioration phase (Austroads 2010a)[3]. Rutting and cracking contribute to the roughness model, with the structural model also contributing to rutting and roughness as illustrated in Figure 5.3.

[3] Whereas the ARRB setup only employed a gradual deterioration phase, evidence exists from the Accelerated Loading Facility Austroads/ARRB studies to inform a higher rate of change of rutting and roughness by a factor of approximately two (Martin, Toole & Oliver 2004) where high cracking and moisture exists. This is a possible further refinement to the basic model for practical application purposes.

Figure 5.3: Interaction between Austroads model components



Source: NACOE project A21, adapted from Austroads (2010a) and Austroads (2010b).

5.1.2 *Comparisons of the MRWA and ARRB model estimates*

Comparisons of the roughness and rutting 'gradual' rates of deterioration derived from applying the MRWA and ARRB models are illustrated in Figure 5.4 and Figure 5.5 respectively. In the examples, two levels of annual lane ESA (measured in millions) of 0.1 MESA and 0.5 MESA were chosen to represent MRWA road link category AW and BW respectively for comparison purposes. These are the dominant road link categories in the TSD 900 dataset. The MRWA deterioration rates in the chart represent the best-case scenario of gradual deterioration of the road link category BW without cracking and the worst-case scenario of road link category AW with minor cracking.

Figure 5.4: MRWA gradual vs Austroads roughness deterioration model

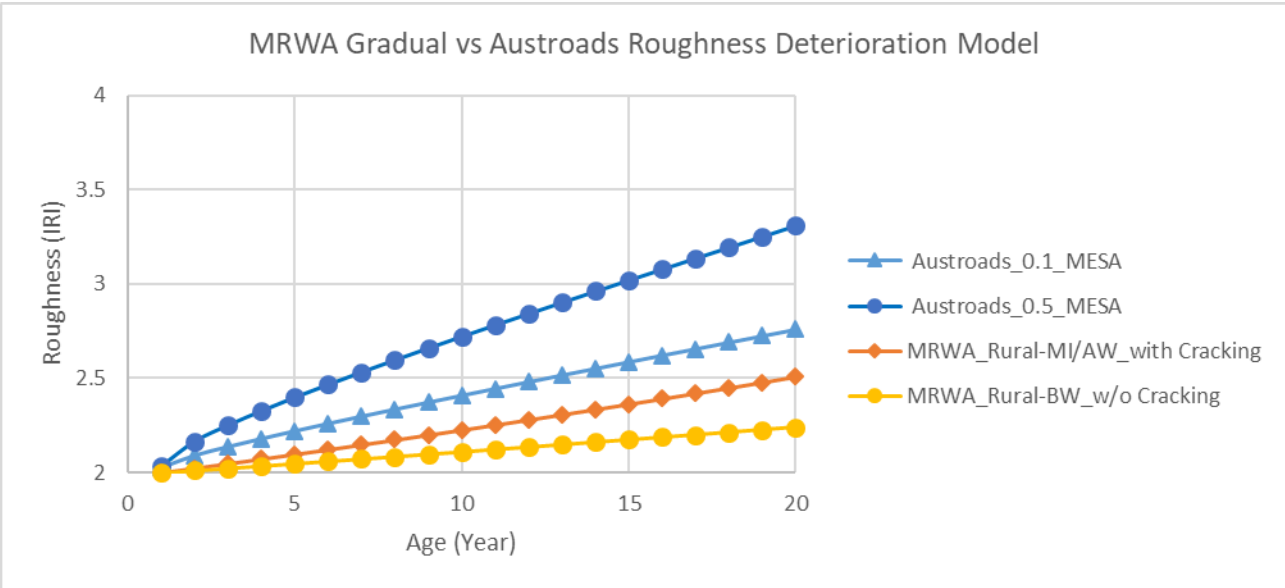
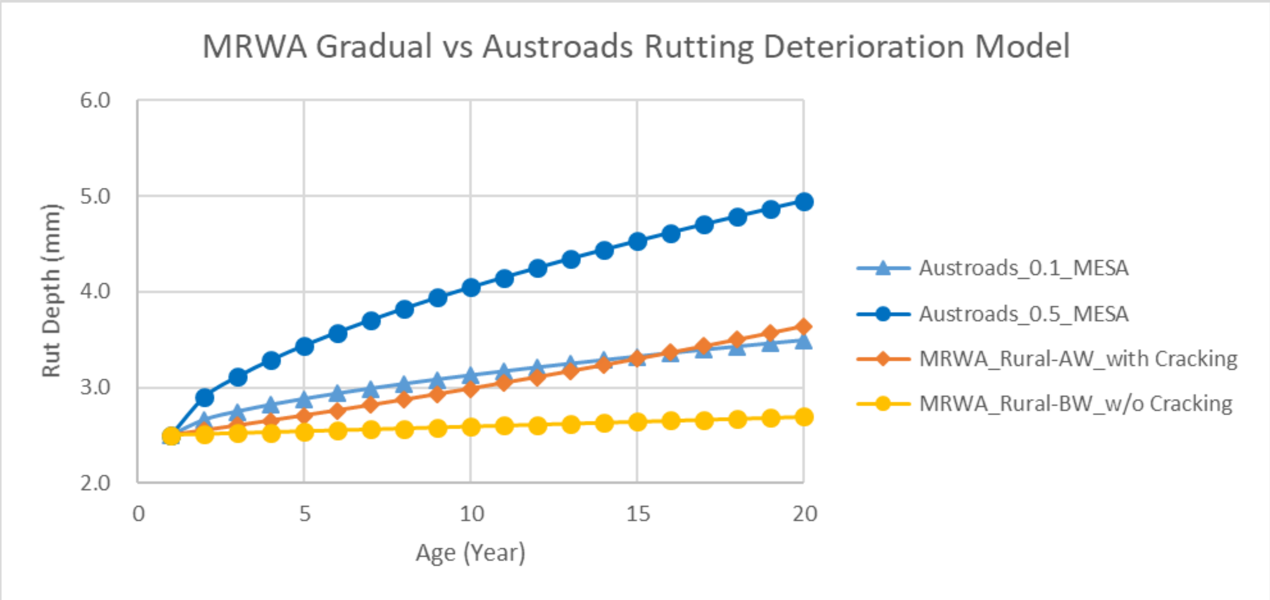


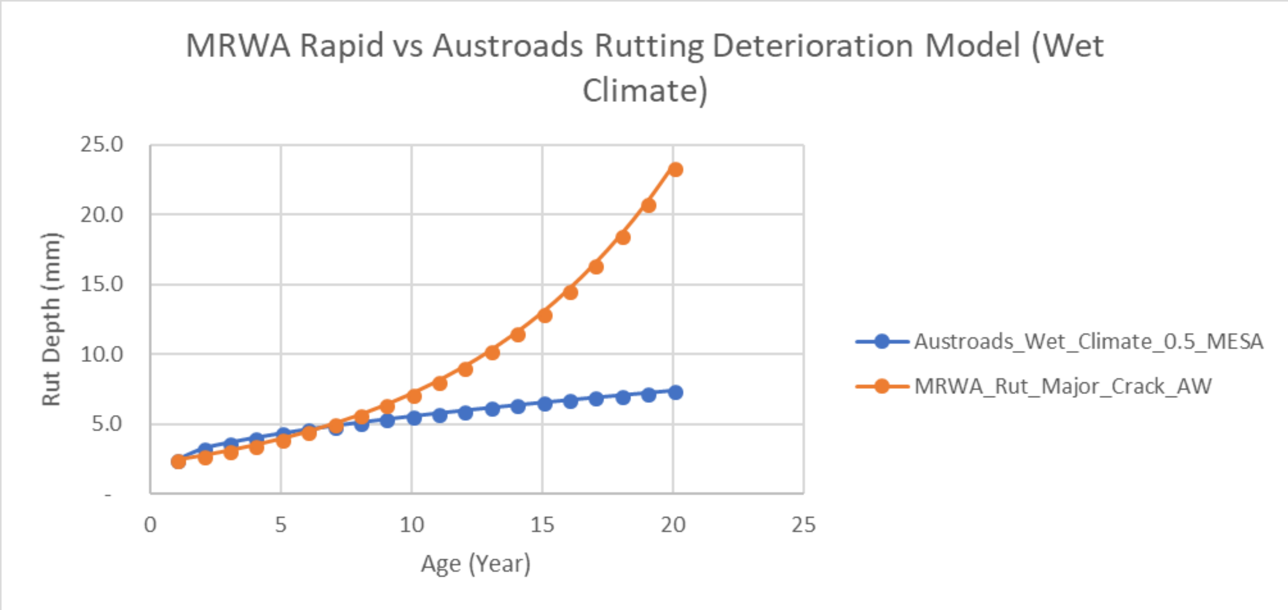
Figure 5.5: MRWA gradual vs Austroads rutting deterioration model



From the above, the Austroads roughness model estimates a more aggressive gradual deterioration rate than that estimated by the MRWA model. A similar trend was also observed when comparing the rutting model. Note, because the Austroads models were not calibrated to the WA conditions, the model coefficients derived from the LTPP project (Austroads 2010a) were used.

In a further example, the Austroads rutting model was applied assuming a wet climate (TMI = 50) with a MESA of 0.5 and compared against the MRWA rapid deterioration model for sprayed seal granular pavement in a high rainfall area (Figure 5.6). As evident, the MRWA model estimates a significant increase in deterioration occurs when a pavement enters the rapid deterioration phase. The model used applies the findings of the Austroads/ARRB ALF studies referred to earlier.

Figure 5.6: Comparison of the MRWA model estimates and the Austroads rutting models for the rapid deterioration phase



5.2 Network level application with dTIMS

5.2.1 *Analysed setups*

Different analysis setups were prepared in dTIMS to allow the results of the various structural modelling methods of assessing pavement structural capacity to be employed, i.e. the Austroads SNC ratio (Method A), the Austroads Notional Structural Life (NSL, Method B) and the ARRB STEP (Method C). All the methods have been discussed separately in Section 3.3. Within each method, two separate cases for estimating the initial structural number have also been trialled. Table 5.1 details the setups analysed.

Table 5.1: dTIMS case study setups

Method	ARRB dTIMS setup code	SNC_0	Treatment trigger
A – Austroads SNC Ratio	ARRB_A1	Estimated using empirical relationship	RSL_SNCratio_est
A – Austroads SNC Ratio	ARRB_A2	Back-calculated SNCi	RSL_SNCratio_bc
B – Austroads NSL	ARRB_B1	Estimated using empirical relationship	RSL_NSL_est
B – Austroads NSL	ARRB_B2	Back-calculated SNCi	RSL_NSL_bc
C – ARRB STEP	ARRB_C2	Back-calculated SNCi	RSL_STEP_bc

Each setup was analysed for:

- an unconstrained budget scenario
- a 20–year analysis period
- a 60–year service life
- optimised for PHI.

5.2.2 *Treatment triggers*

The treatments and associated triggers adopted in the ARRB dTIMS setup mimicked those of MRWA's with the following adjustments:

- Instead of curvature and deflection parameters, a remaining structural life (RSL) value was used as the 'basic' structural parameter. In the case of the SNC ratio, a trigger of 0.59 was used. The model was used to convert the SNC ratio at the time of survey to the trigger ratio expressed in RSL. The NSL and STEP were both expressed in terms of RSL.
- The Austroads cracking model was used, and data was expressed as the percentage of cracking instead of the crack score.

The resulting triggers are illustrated in Table 5.2 for a typical MRWA dTIMS setup and in Table 5.3 for the ARRB dTIMS setup. The newly introduced or modified condition parameters are highlighted in yellow.

For a full set of the modified treatment triggers refer to Appendix B. The example in Appendix B is showing a typical treatment trigger set for Method A only but is applicable for Methods B and C as well since all three methods are using the same trigger level of RSL ratio of 0.59.

Table 5.2: Example treatment trigger from MRWA pavement modelling manual

Treatment		Curv	Defl (micron)	Surf Age	Rgh (IRI)	Rutting	Crack (Score)	SurfType	MMIS Defect Intensity ($/km)
Sprayed Seal (CS)	Basic	<300	≥800	>2	<3.44	<15	<2	Not Asphalt	<90k
	Trigger			>Max Age					
	Basic		<800	>2		<15		Not Asphalt	<90k
	Trigger			>Max Age			2		

Table 5.3: Example treatment trigger in ARRB dTIMS (SNC ratio as structural capacity indicator)

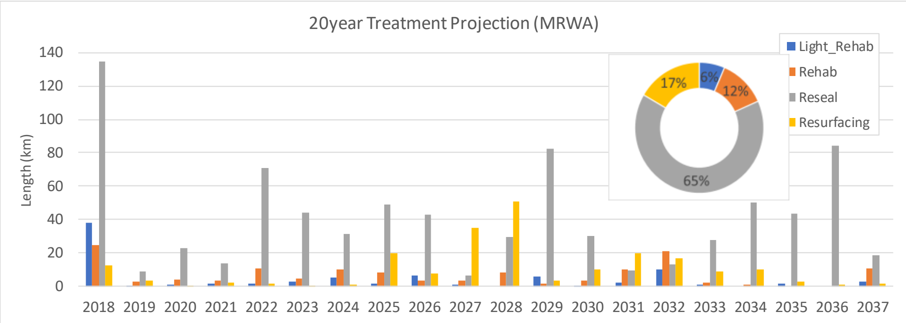
Treatment			RSL ratio	Surf Age	Rgh (IRI)	Rutting	Crack Extent (%)	SurfType	MMIS Defect Intensity ($/km)
Sprayed Seal (CS)	Basic		<0.59	>2	<3.82	<15	<25	Not Asphalt	<90k
	Trigger			>Max Age					
	Basic		≥0.59	>2		<15		Not Asphalt	<90k
	Trigger			>Max Age			≥25		

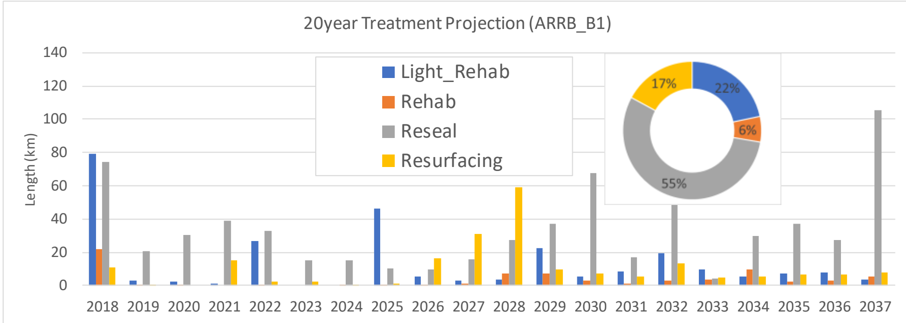
5.3 Analysis Results

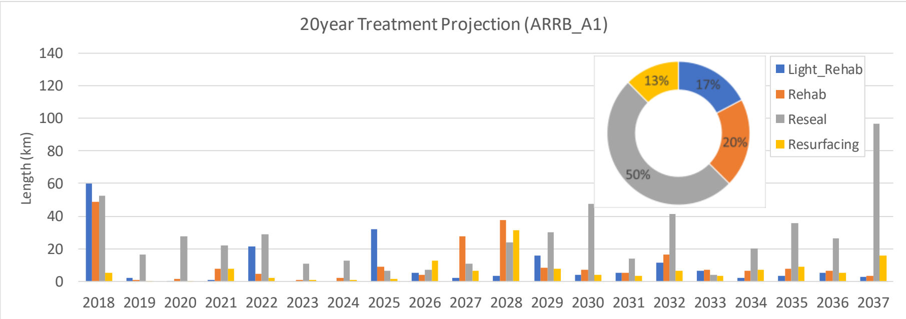
Several MRWA dTIMS runs were completed for the project employing different data input. Comparisons were made with the results from the various ARRB setups, ARRB_A1, A2, B1, B2 and C2. All runs used the same data input based on the averaged aggregation from 100 m to 500 m sections.

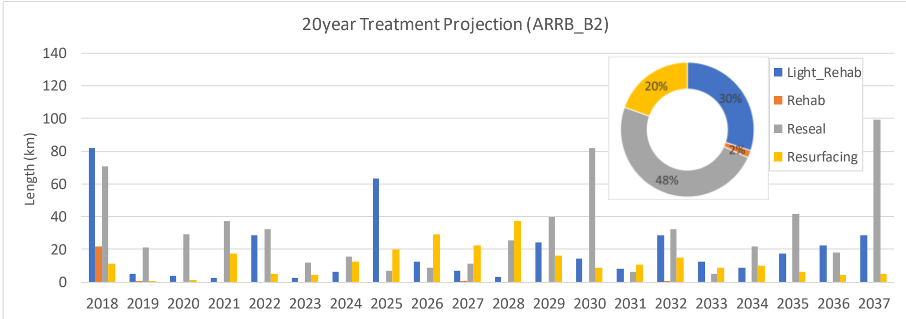
The treatments have been grouped into treatment classes of light rehabilitation, rehabilitation, reseal and resurfacing for comparison purposes. For each dTIMS setup, the sum of the length of a treatment class triggered over the 20–year analysis period is presented in Figure 5.7.

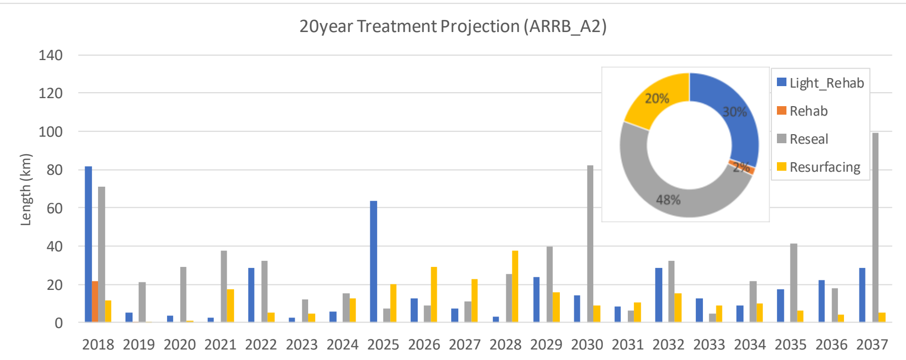
Figure 5.7: Treatment triggered distribution over 20 year period by length

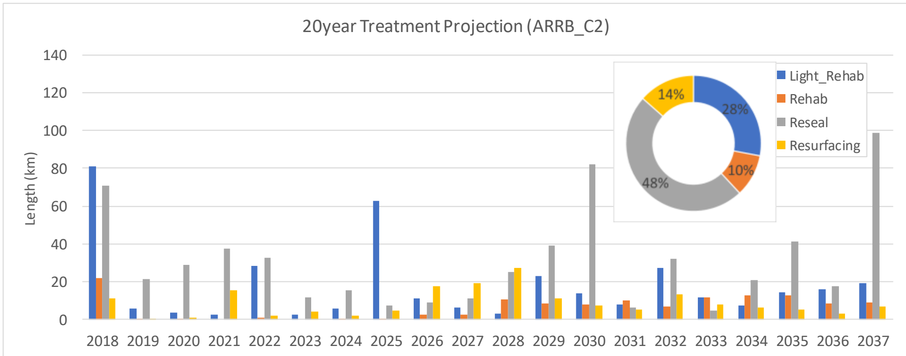

20year Treatment Projection (MRWA)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
6%
12%
65%
17%


20year Treatment Projection (ARRB_B1)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
22%
6%
55%
17%


20year Treatment Projection (ARRB_A1)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
17%
20%
50%
13%

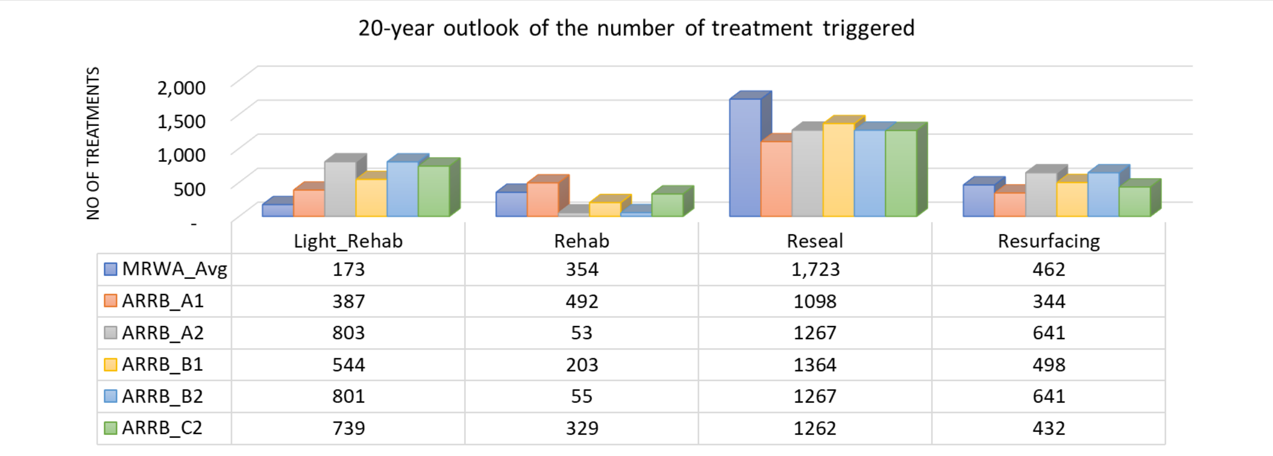

20year Treatment Projection (ARRB_B2)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
30%
2%
48%
20%


20year Treatment Projection (ARRB_A2)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
30%
2%
48%
20%


20year Treatment Projection (ARRB_C2)
Length (km)
140
120
100
80
60
40
20
0
2018 2019 2020 2021 2022 2023 2024 2025 2026 2027 2028 2029 2030 2031 2032 2033 2034 2035 2036 2037
Light_Rehab
Rehab
Reseal
Resurfacing
28%
10%
48%
14%

A summary of the sum of the treatments triggered for each dTIMS setup is provided in Figure 5.8 and the total treatment cost for each setup is summarised in Table 5.4.

Figure 5.8: A 20–year outlook of number of treatments triggered



	Light_Rehab	Rehab	Reseal	Resurfacing
MRWA_Avg	173	354	1,723	462
ARRB_A1	387	492	1098	344
ARRB_A2	803	53	1267	641
ARRB_B1	544	203	1364	498
ARRB_B2	801	55	1267	641
ARRB_C2	739	329	1262	432

Table 5.4: Total spending in various treatment classes

Treatment	MRWA	ARRB_A1	ARRB_A2	ARRB_B1	ARRB_B2	ARRB_C2
Light rehab	$33 815 000	$72 963 000	$151 4670 000	$101 274 000	$151 293 000	$140 431 000
Rehab	$110 067 000	$194 785 000	$13 093 000	$67 677 000	$13 352 000	$118 105 000
Reseal	$34 897 000	$17 980 000	$20 922 000	$22 518 000	$20 923 000	$20 855 000
Resurfacing	$66 889 000	$53 511 000	$105 309 000	$86 107 000	$105 309 000	$73 286 000
Total	$245 670 116	$339 240 000	$290 795 000	$277 576 000	$290 877 000	$352 678 000

Observations on the results are as follow:

- The results of setup ARRB_A2 and ARRB_B2 are almost identical in terms of the total cost as well as when treatments were triggered.
- A very small proportion of rehabilitation treatment is triggered when SNC_0 was back-calculated as observed for the analysis result of ARRB_A2 and ARRB_B2. Most of these were triggered in the first year and not because of the structural parameter but from the MMIS criteria. Because of the slow structural deterioration prediction of these two methods, no other rehabilitation treatment was triggered throughout the 20–year analysis period. Instead, only light rehabilitation, reseal or resurfacing treatments were triggered. Consequently, in this example for the TSD 900 sub-network, the ARRB_A2 and ARRB_B2 setups are not affected by the structural deterioration.
- The MRWA dTIMS setup appears to promote a preservation strategy of reseal or resurfacing over the selection of costlier rehabilitation treatments. Compared to the other setups, this is the lowest cost setup under the unconstrained budget scenario. This is evident in the first-year needs which are dominated by resealing or resurfacing. However, the lengths of light rehabilitation treatments triggered are significantly less than the other setups especially in the first year. The ARRB setups adopted a more aggressive functional model which resulted in more frequent occurrences of light rehabilitation in the later years.

- The length of the light rehabilitation triggered in the first year for the analysis result of ARRB_A2, B1, B2 and C2 are higher than ARRB_A1 and more than double that of MRWA's. A closer inspection of the data indicates that the need for more light rehabilitation in the first year is warranted with most triggered by high roughness. A field investigation is required to validate the need for light rehabilitation.
- The ARRB_A1 setup produces the highest lengths of rehabilitation treatment not only in the first year but also in later years, notably in 2027 onwards. The MRWA setup and ARRB_C2, although not as much as ARRB_A1, also generate significant lengths of rehabilitation in the later years. The rehabilitation needs in the first year appear to be warranted, i.e. poor functional condition with high deflection and curvature. A field validation is required to confirm needs.
- The need for more rehabilitation generated from the ARRB_A1 setup in the later years is driven by the remaining structural life expectation on certain pavement types as shown in Figure 3.6. In this example, asphalt on stabilised pavement gives the shortest remaining life expectation followed by sprayed seal on unbound pavements. Consequently, the spike in the need for rehabilitation around 2027 and 2028 are all on thin asphalt surfacing on stabilised pavements, whereas rehabilitation needs on the sprayed seal on stabilised material are predicted as being required much later. To confirm these predictions, or otherwise, long-term performance monitoring is recommended.
- The setup that produces the costliest work program over the 20–year period is ARRB_C2.
- Where funding is constrained, the potential backlog is reflected in the first-year type and quantum of works as shown in Figure 5.7. The MRWA backlog results are dominated by reseal/resurfacing work whereas the other setups predict a need for a substantial amount of light rehabilitation as the backlog.
- The first-year work program is also an indication of how each setup assesses the current sub-network condition. All ARRB setups except for ARRB_A1 show a similar type and quantum of works. This could be a useful starting point when validating the result with field investigations.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary of findings

The project set out to maximise the use of the pavement strength data collected with the TSD in 2016 over approximately 900 km of the MRWA sub-network, TSD 900. The survey was conducted by ARRB who provided TSD survey output which utilised the area under the curve method to estimate deflection parameters based on Muller and Roberts (2012). The TSD deflection was converted to equivalent FWD deflection parameters using the relationship developed under NACOE in Queensland (Lee 2016). It is understood that a similar study to establish a TSD-FWD relationship is currently underway in MRWA, and this should be used once available.

A review of the international and domestic practices on determining pavement structural capacity was carried out. Three methods of estimating a pavement structural index were identified and compared using the data from the TSD 900. The comparison used the Microsoft Power BI platform to plot the remaining structural life (RSL) derived from the TSD deflections by the three methods as well as the functional conditions. This, together with the available video allowed the team to conduct a limited validation of the analysis results. Two variations of Method A, Austroads SNC ratio with back-calculated SNC_0 and the other with an estimated SNC_0, Method B being the adapted Austroads Notional Structural Life and Method C the ARRB STEP were compared.

The RSL estimates calculated with Method A which estimated SNC_0 and Method C fluctuate considerably with the change in deflection and pavement type. The estimated RSL was shown to be shorter for both methods when the base material is stabilised. It was also the shortest when the pavement type comprised thin asphalt on a stabilised pavement. Method A with the back-calculated SNC_0 and Method B produced a constantly high value of RSL with the former showing little response to fluctuations in measured deflections.

Environmental and geological factors were investigated to determine if they influence structural deterioration. The TMI, temperature and precipitation were used as the environmental factors and the geological factor was represented by soil-type groups represented in the WA soil classification. A statistical correlation analysis was carried out using the RSL from Method C as the dependent variable. None of the factors investigated were found to have a significant correlation. However, precipitation was found to produce a better correlation than TMI or temperature.

A final set of case studies were carried out by applying the TSD data in a pavement management system environment, with dTIMS selected as the preferred platform. Five combinations of the ARRB dTIMS setup were employed with these using different structural index methods to a common ARRB dTIMS template which utilised the Austroads functional models. Treatment rules and triggers generally followed those specified by MRWA with the exception of the replacement of deflection and curvature parameters employed in the MRWA setup with RSL. MRWA separately ran their own dTIMS setup and provided the results to ARRB. The analysis results were compared by examining the length and type of treatments triggered over the 20–year analysis period, as well as estimated treatment costs, with the following conclusions drawn:

- Results from ARRB_A2 and ARRB_B2 are almost identical in terms of the total cost as well as when treatments are triggered, with neither setup significantly affected by structural deterioration during the 20–year analysis period.
- The lowest treatment cost estimate was generated from the MRWA setup, which is dominated by preservation treatments (reseal/resurfacing) over rehabilitation early in the analysis period.

- The setup that produced the costliest work program over the 20–year period was ARRB_C2.

The project also considered the use of 10 metre interval structural data to address MRWA needs to identify ‘pavement repairs’ prior to resealing. A prototype of a dynamic segmentation tool was developed to allow MRWA a quick scan of any road for potential pavement related issues.

6.2 Recommendations for follow-up studies

The following recommendations are provided related to the continuation of this research:

1. There is a significant difference in the roughness and rutting model estimates from applying the Austroads/ARRB and MRWA models and this is reflected in the analysis results[4]. This should be addressed through a comprehensive calibration exercise based on time-series data to ensure the models match the actual network performance.
2. A wider sample, including other pavement configurations covering a broader range of climate zones and other factors such as drainage condition, pavement age and updated soil information should also be investigated.
3. Whilst roughness and rutting have been well integrated in MRWA pavement modelling, cracking data is not yet fully utilised. MRWA should investigate adopting an incremental cracking model such as the Austroads cracking model instead of relying on annual cracking scores assessed by the region.
4. The immediate validation by means of field investigation of the analysis results using the first-year work programs is highly recommended to confirm the accuracy of the setups. However, to assess the prediction of future needs, a combination of a calibration exercise using historical condition data and a long-term monitoring program is recommended. With the TSD, MRWA is now able to obtain more precise functional and structural data and should take advantage of its availability.
5. Further work should aim to enhance the current MRWA dTIMS setup by taking advantage of the finer detail from the TSD data to identify potential structural issues, i.e. ‘pavement repairs’ as input to works programming and costing.
6. The conversion of TSD measured deflection to the FWD equivalent should use the relationship from the Western Australia study when it is available.

[4] Since the completion of this study MRWA has re-estimated road deterioration models for WA, and the results are reasonably consistent with the Austroads/ARRB models. The implication of this is both budget and condition estimates should therefore be similar, but this can only be confirmed by rerunning the MRWA dTIMS setup.

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APPENDIX A SUMMARY OF DATA RECEIVED

Table A 1: Summary of data received and used

No	Category	Data	Type	Provided by	Coverage	Comments
1	Climate	TMI	Data	Calculated by ARRB	All 900 km is within −40 to −50 (arid climate zone)	Main climate zone based on TMI
2	Pavement Configuration	PAOR – Base, Sub-base (type, thickness, year etc.)	Data	MRWA	For H005 & H003 only	Not used
3	Pavement configuration	PAOR – Base, Sub-base (type, thickness, year etc.)	Data	MRWA	900 km	Used as the secondary pavement inventory data
4	Surfacing information	SULA – (type, agg. size, year etc.)	Data	MRWA	For H005 & H003 only	Used as the secondary surfacing inventory data
5	TSD	Measured deflection bowl with TSD, cracking, roughness, rutting, @10 m interval	Data	ARRB system	900 km	Used as main functional and structural condition
6	TSD	Raw survey file (video & data)	Video & data	ARRB system	900 km	Used as the secondary validation tool
7	TSD	Measured deflection bowl with TSD – extracted from IRIS – no cracking	Data	MRWA	Does not include functional condition	Used as the base to align ARRB TSD survey and MRWA referencing
8	MMIS	Open and closed defect from MRWA MMIS	Data	MRWA	For H005 & H003 only	Used as validation data
9	Visual inspection	Pavement condition rating 1–4 as recently rated by Kyran	Data	MRWA	For H005 & H003 only	Used as validation data
10	Tableau input	Collection of Inventory, condition, traffic, MMIS (lane/km) cost, @20 m interval	Data	MRWA	Entire network	Used as main traffic source data and the main pavement and surfacing inventory data.
11	Traffic data	MRWA traffic map	Data	MRWA	Entire network in an interactive web-based map	Not used
12	Soil classification	WA Gov map of soil alkalinity, subsurface, landscape etc.	Data	MRWA	GIS map based	Not used
13	Soil classification	State Road Soil System	Data	MRWA	Tabular information with SLK and type of soil system	Reclassified into a simplified category – used as main soil classification data
14	Tableau manual	Documented manual in using tableau –pPavement condition data analytics	Document	MRWA		
15	MRWA dTIMS setup	Documented procedure/rules adopted – pavement modelling spec	Document	MRWA		Used to reconfigure ARRB dTIMS setup to match MRWA treatment

No	Category	Data	Type	Provided by	Coverage	Comments
16	ARRB dTIMS setup	Documented procedure/rules/models adopted – VicRoads/QTMR setup?	Document	ARRB		Used to reconfigure ARRB dTIMS setup to match MRWA treatment
17	dTMS file	The latest dTIMS output	Data	MRWA		Not used
18	dTMS file	A sample input file used in dTIMS – LCC	Data	MRWA	900 km	Revised to include ARRB deflection and curvature and sent back to MRWA

APPENDIX B ARRB TREATMENT TRIGGER EXAMPLE FOR AW ROAD CLASS

Table B 1: Modified treatment trigger for AW road class

Treatment			RSL ratio	Surf Age	Rgh (IRI)	Rutting	Crack Extent (%)	SurfType	MMIS Defect Intensity ($/km)	Posted Speed
Sprayed Seal (CS)	Basic		<0.59	>2	<3.82	<15	<25	Not Asphalt	<90k	
	Trigger			>Max Age						
	Basic		≥0.59	>2		<15		Not Asphalt	<90k	
	Trigger			>Max Age			≥25			
Slurry	Basic		≥0.59	>Max Age		<15	<25	Not Asphalt	<90k	
	Trigger					≥10				
Light Rehab (RipSeal)	Basic		≥0.59	>4				Not Asphalt	<90k	
	Trigger				≥3.82	≥15				
Heavy Rehab (GrOL)	Basic		<0.59	>4				Not Asphalt		
	Trigger				≥3.82	≥15				
	Basic			>4				Not Asphalt		
	Trigger								≥90k	
Holding Reseal (HCS)	Basic		<0.59	>4				Not Asphalt		
	Trigger				≥3.82		≥25			
	Basic			>4				Not Asphalt		
	Trigger					≥15			≥90k	
Dense Graded Asphalt (ASDG)	Basic		≥0.59	>4				DGA		<90
	Trigger			>Max Age	≥3.82	≥15	≥25			
Intersection Mix Asphalt (ASIM)	Basic		≥0.59	>4				IMA		<90
	Trigger			>Max Age	≥3.82	≥15	≥25			
Open Graded Asphalt (ASOG)	Basic		≥0.59	>4				OGA/OGA2		
	Trigger			>Max Age	≥3.82	≥15				
	Basic		≥0.59	>8				OGA/OGA2		
	Trigger						≥25			
OGA on DGA (OGA2)	Basic		≥0.59	<=8				OGA/OGA2		
	Trigger						≥25			
Stone Mastic Asphalt (SMA)	Basic		≥0.59	>4				SMA		
	Trigger			>Max Age	≥3.82	≥15	≥25			
	Basic		≥0.59	>4				DGA. IMA		≥90
	Trigger			>Max Age	≥3.82	≥15	≥25			
Structural Asphalt (ASRH)	Basic		<0.59	>4				Asphalt		
	Trigger			>Max Age	≥3.82	≥15	≥25			
	Basic			>4				Asphalt		
	Trigger								≥90k	

APPENDIX C SOIL CLASSIFICATION FOR THE PROJECT

Table C 1: Soil classification translation table

MUSuperGps_NAME	Soil_Group	Soil_Class
Cracking clays supergroup	CC_	Clay
Cracking clays supergroup & Loamy earths supergroup	CCL	Clay_Loam
Cracking clays supergroup, Sandy duplexes supergroup, deep & Non-cracking clays supergroup	NCCCS	Clay_Sand
Deep sands supergroup	S	Sand
Deep sands supergroup & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Deep sands supergroup & Loamy earths supergroup	LS	Loam_Sand
Deep sands supergroup & Non-cracking clays supergroup	NCCCS	Clay_Sand
Deep sands supergroup & Sandy earths supergroup	S	Sand
Deep sands supergroup & Shallow loams supergroup	LS	Loam_Sand
Deep sands supergroup & Shallow sands supergroup	S	Sand
Deep sands supergroup & Wet or waterlogged soils supergroup	SW	Sand_Waterlogged
Deep sands supergroup, Ironstone gravelly soils supergroup & Wet or waterlogged soils supergroup	SGW	Sand_Gravel_Waterlogged
Deep sands supergroup, Loamy duplexes supergroup, deep & Sandy duplexes supergroup, deep	LS	Loam_Sand
Deep sands supergroup, Loamy duplexes supergroup, shallow & Ironstone gravelly soils supergroup	LSG	Loam_Sand_Gravel
Deep sands supergroup, Rocky or stony soils supergroup & Sandy earths supergroup	SR	Sand_Rocky
Deep sands supergroup, Sandy duplexes supergroup, deep & Loamy earths supergroup	LS	Loam_Sand
Deep sands supergroup, Sandy earths supergroup & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Deep sands supergroup, Sandy earths supergroup & Shallow loams supergroup	LS	Loam_Sand
Deep sands supergroup, Wet or waterlogged soils supergroup & Sandy duplexes supergroup, deep	SW	Sand_Waterlogged
Ironstone gravelly soils supergroup	G	Gravel
Ironstone gravelly soils supergroup & Loamy earths supergroup	LG	Loam_Gravel
Ironstone gravelly soils supergroup & Rocky or stony soils supergroup	RG	Gravel_Rocky
Ironstone gravelly soils supergroup & Sandy duplexes supergroup, deep	SG	Sand_Gravel
Ironstone gravelly soils supergroup & Wet or waterlogged soils supergroup	GW	Gravel_Waterlogged
Ironstone gravelly soils supergroup, Deep sands supergroup & Sandy duplexes supergroup, deep	SG	Sand_Gravel
Ironstone gravelly soils supergroup, Deep sands supergroup & Sandy duplexes supergroup, shallow	SG	Sand_Gravel
Ironstone gravelly soils supergroup, Deep sands supergroup & Wet or waterlogged soils supergroup	SGW	Sand_Gravel_Waterlogged
Ironstone gravelly soils supergroup, Loamy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	LSG	Loam_Sand_Gravel

MUSuperGps_NAME	Soil_Group	Soil_Class
Ironstone gravelly soils supergroup, Loamy earths supergroup & Loamy duplexes supergroup, deep	LG	Loam_Gravel
Ironstone gravelly soils supergroup, Loamy earths supergroup & Rocky or stony soils supergroup	LRG	Loam_Rocky_Gravel
Ironstone gravelly soils supergroup, Rocky or stony soils supergroup & Sandy duplexes supergroup, deep	SRG	Sand_Rocky_Gravel
Ironstone gravelly soils supergroup, Sandy duplexes supergroup, deep & Sandy duplexes supergroup, shallow	SG	Sand_Gravel
Ironstone gravelly soils supergroup, Sandy duplexes supergroup, deep & Wet or waterlogged soils supergroup	SGW	Sand_Gravel_Waterlogged
Ironstone gravelly soils supergroup, Sandy duplexes supergroup, shallow & Loamy duplexes supergroup, shallow	LSG	Loam_Sand_Gravel
Ironstone gravelly soils supergroup, Sandy duplexes supergroup, shallow & Loamy earths supergroup	LSG	Loam_Sand_Gravel
Ironstone gravelly soils supergroup, Sandy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	SG	Sand_Gravel
Ironstone gravelly soils supergroup, Wet or waterlogged soils supergroup & Deep sands supergroup	SGW	Sand_Gravel_Waterlogged
Loamy duplexes supergroup, deep	L	Loam
Loamy duplexes supergroup, deep, Loamy duplexes supergroup, shallow & Rocky or stony soils supergroup	LR	Loam_Rocky
Loamy duplexes supergroup, shallow	L	Loam
Loamy duplexes supergroup, shallow & Cracking clays supergroup	CCL	Clay_Loam
Loamy duplexes supergroup, shallow & Loamy earths supergroup	L	Loam
Loamy duplexes supergroup, shallow & Sandy duplexes supergroup, shallow	LS	Loam_Sand
Loamy duplexes supergroup, shallow, Ironstone gravelly soils supergroup & Loamy earths supergroup	LG	Loam_Gravel
Loamy duplexes supergroup, shallow, Loamy earths supergroup & Wet or waterlogged soils supergroup	LW	Loam_Waterlogged
Loamy duplexes supergroup, shallow, Sandy duplexes supergroup, deep & Ironstone gravelly soils supergroup	LSG	Loam_Sand_Gravel
Loamy duplexes supergroup, shallow, Sandy duplexes supergroup, deep & Rocky or stony soils supergroup	LSR	Loam_Sand_Rocky
Loamy duplexes supergroup, shallow, Sandy duplexes supergroup, shallow & Deep sands supergroup	LS	Loam_Sand
Loamy duplexes supergroup, shallow, Sandy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	LS	Loam_Sand
Loamy earths supergroup	L	Loam
Loamy earths supergroup & Deep sands supergroup	LS	Loam_Sand
Loamy earths supergroup & Ironstone gravelly soils supergroup	LG	Loam_Gravel
Loamy earths supergroup & Non-cracking clays supergroup	NCCCL	Clay_Loam
Loamy earths supergroup & Rocky or stony soils supergroup	LR	Loam_Rocky
Loamy earths supergroup & Sandy earths supergroup	LS	Loam_Sand

MUSuperGps_NAME	Soil_Group	Soil_Class
Loamy earths supergroup & Shallow loams supergroup	L	Loam
Loamy earths supergroup, Cracking clays supergroup & Loamy duplexes supergroup, shallow	CCL	Clay_Loam
Loamy earths supergroup, Ironstone gravelly soils supergroup & Loamy duplexes supergroup, shallow	LG	Loam_Gravel
Loamy earths supergroup, Loamy duplexes supergroup, shallow & Loamy duplexes supergroup, deep	L	Loam
Loamy earths supergroup, Sandy duplexes supergroup, shallow & Shallow loams supergroup	LS	Loam_Sand
Loamy earths supergroup, Sandy earths supergroup & Ironstone gravelly soils supergroup	LSG	Loam_Sand_Gravel
Loamy earths supergroup, Shallow loams supergroup & Sandy duplexes supergroup, shallow	LS	Loam_Sand
Loamy earths supergroup, Shallow sands supergroup & Shallow loams supergroup	LS	Loam_Sand
Loamy earths supergroup, Wet or waterlogged soils supergroup & Sandy duplexes supergroup, deep	LSW	Loam_Sand_Waterlogged
Loamy earths supergroup, Wet or waterlogged soils supergroup & Sandy duplexes supergroup, shallow	LSW	Loam_Sand_Waterlogged
Miscellaneous soils supergroup		#N/A
Non-cracking clays supergroup	NCCC	Clay
Non-cracking clays supergroup & Cracking clays supergroup	NCCC	Clay
Non-cracking clays supergroup & Loamy duplexes supergroup, deep	NCCCL	Clay_Loam
Non-cracking clays supergroup & Sandy duplexes supergroup, shallow	NCCCS	Clay_Sand
Non-cracking clays supergroup & Shallow loams supergroup	NCCCL	Clay_Loam
Non-cracking clays supergroup, Cracking clays supergroup & Loamy duplexes supergroup, deep	NCCCL	Clay_Loam
Non-cracking clays supergroup, Loamy earths supergroup & Loamy duplexes supergroup, deep	NCCCL	Clay_Loam
Non-cracking clays supergroup, Sandy duplexes supergroup, deep & Sandy duplexes supergroup, shallow	NCCCS	Clay_Sand
Rocky or stony soils supergroup	R	Rocky
Rocky or stony soils supergroup & Deep sands supergroup	SR	Sand_Rocky
Rocky or stony soils supergroup & Loamy duplexes supergroup, shallow	LR	Loam_Rocky
Rocky or stony soils supergroup & Sandy duplexes supergroup, shallow	SR	Sand_Rocky
Rocky or stony soils supergroup & Sandy earths supergroup	SR	Sand_Rocky
Rocky or stony soils supergroup & Shallow loams supergroup	LR	Loam_Rocky
Rocky or stony soils supergroup & Shallow sands supergroup	SR	Sand_Rocky
Rocky or stony soils supergroup, Ironstone gravelly soils supergroup & Loamy earths supergroup	LRG	Loam_Rocky_Gravel
Rocky or stony soils supergroup, Loamy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	LSR	Loam_Sand_Rocky
Rocky or stony soils supergroup, Shallow sands supergroup & Loamy earths supergroup	LSR	Loam_Sand_Rocky

MUSuperGps_NAME	Soil_Group	Soil_Class
Sandy duplexes supergroup, deep	S	Sand
Sandy duplexes supergroup, deep & Deep sands supergroup	S	Sand
Sandy duplexes supergroup, deep & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Sandy duplexes supergroup, deep & Loamy earths supergroup	LS	Loam_Sand
Sandy duplexes supergroup, deep & Non-cracking clays supergroup	NCCCS	Clay_Sand
Sandy duplexes supergroup, deep & Sandy duplexes supergroup, shallow	S	Sand
Sandy duplexes supergroup, deep & Shallow loams supergroup	LS	Loam_Sand
Sandy duplexes supergroup, deep, Deep sands supergroup & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Sandy duplexes supergroup, deep, Deep sands supergroup & Loamy duplexes supergroup, shallow	LS	Loam_Sand
Sandy duplexes supergroup, deep, Ironstone gravelly soils supergroup & Loamy duplexes supergroup, shallow	LSG	Loam_Sand_Gravel
Sandy duplexes supergroup, deep, Ironstone gravelly soils supergroup & Sandy duplexes supergroup, shallow	SG	Sand_Gravel
Sandy duplexes supergroup, deep, Loamy duplexes supergroup & Loamy earths supergroup	LS	Loam_Sand
Sandy duplexes supergroup, deep, Loamy duplexes supergroup, deep & Ironstone gravelly soils supergroup	LSG	Loam_Sand_Gravel
Sandy duplexes supergroup, deep, Loamy duplexes supergroup, deep & Non-cracking clays supergroup	NCCCLS	Clay_Loam_Sand
Sandy duplexes supergroup, deep, Loamy duplexes supergroup, deep & Shallow sands supergroup	LS	Loam_Sand
Sandy duplexes supergroup, deep, Loamy earths supergroup & Cracking clays supergroup	CCLS	Clay_Loam_Sand
Sandy duplexes supergroup, deep, Sandy duplexes supergroup, shallow & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Sandy duplexes supergroup, deep, Sandy duplexes supergroup, shallow & Shallow sands supergroup	S	Sand
Sandy duplexes supergroup, deep, Shallow sands supergroup & Sandy duplexes supergroup, shallow	S	Sand
Sandy duplexes supergroup, deep, Wet or waterlogged soils supergroup & Deep sands supergroup	SW	Sand_Waterlogged
Sandy duplexes supergroup, deep, Wet or waterlogged soils supergroup & Ironstone gravelly soils supergroup	SGW	Sand_Gravel_Waterlogged
Sandy duplexes supergroup, shallow	S	Sand
Sandy duplexes supergroup, shallow & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Sandy duplexes supergroup, shallow & Loamy duplexes supergroup, shallow	LS	Loam_Sand
Sandy duplexes supergroup, shallow & Loamy earths supergroup	LS	Loam_Sand
Sandy duplexes supergroup, shallow & Rocky or stony soils supergroup	SR	Sand_Rocky
Sandy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	S	Sand
Sandy duplexes supergroup, shallow & Shallow loams supergroup	LS	Loam_Sand

MUSuperGps_NAME	Soil_Group	Soil_Class
Sandy duplexes supergroup, shallow & Shallow sands supergroup	S	Sand
Sandy duplexes supergroup, shallow, Deep sands supergroup & Sandy earths supergroup	S	Sand
Sandy duplexes supergroup, shallow, Ironstone gravelly soils supergroup & Loamy duplexes supergroup, shallow	LSG	Loam_Sand_Gravel
Sandy duplexes supergroup, shallow, Loamy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	LS	Loam_Sand
Sandy duplexes supergroup, shallow, Sandy duplexes supergroup, deep & Loamy duplexes supergroup, deep	LS	Loam_Sand
Sandy duplexes supergroup, shallow, Sandy duplexes supergroup, deep & Loamy duplexes supergroup, shallow	LS	Loam_Sand
Sandy duplexes supergroup, shallow, Shallow loams supergroup & Cracking clays supergroup	CCLS	Clay_Loam_Sand
Sandy duplexes supergroup, shallow, Wet or waterlogged soils supergroup & Loamy earths supergroup	LSW	Loam_Sand_Waterlogged
Sandy duplexes supergroup, shallow, Wet or waterlogged soils supergroup & Sandy duplexes supergroup, deep	SW	Sand_Waterlogged
Sandy earths supergroup	S	Sand
Sandy earths supergroup & Deep sands supergroup	S	Sand
Sandy earths supergroup & Loamy duplexes supergroup	LS	Loam_Sand
Sandy earths supergroup & Loamy earths supergroup	LS	Loam_Sand
Sandy earths supergroup & Sandy duplexes supergroup, shallow	S	Sand
Sandy earths supergroup, Deep sands supergroup & Ironstone gravelly soils supergroup	SG	Sand_Gravel
Sandy earths supergroup, Deep sands supergroup & Sandy duplexes supergroup, deep	S	Sand
Sandy earths supergroup, Ironstone gravelly soils supergroup & Sandy duplexes supergroup, deep	SG	Sand_Gravel
Sandy earths supergroup, Loamy earths supergroup & Loamy duplexes supergroup, shallow	LS	Loam_Sand
Sandy earths supergroup, Shallow sands supergroup & Deep sands supergroup	S	Sand
Shallow loams supergroup	L	Loam
Shallow loams supergroup & Deep sands supergroup	LS	Loam_Sand
Shallow loams supergroup & Loamy earths supergroup	L	Loam
Shallow loams supergroup & Non-cracking clays supergroup	NCCCL	Clay_Loam
Shallow loams supergroup & Rocky or stony soils supergroup	LR	Loam_Rocky
Shallow loams supergroup & Sandy duplexes supergroup, deep	LS	Loam_Sand
Shallow loams supergroup & Shallow sands supergroup	LS	Loam_Sand
Shallow loams supergroup, Loamy earths supergroup & Sandy earths supergroup	LS	Loam_Sand
Shallow loams supergroup, Loamy earths supergroup & Shallow sands supergroup	LS	Loam_Sand
Shallow loams supergroup, Non-cracking clays supergroup & Cracking clays supergroup	NCCCL	Clay_Loam
Shallow loams supergroup, Sandy duplexes supergroup, shallow & Shallow sands supergroup	LS	Loam_Sand

MUSuperGps_NAME	Soil_Group	Soil_Class
Shallow loams supergroup, Shallow sands supergroup & Deep sands supergroup	LS	Loam_Sand
Shallow loams supergroup, Shallow sands supergroup & Sandy duplexes supergroup, shallow	LS	Loam_Sand
Shallow sands supergroup	S	Sand
Shallow sands supergroup & Shallow loams supergroup	LS	Loam_Sand
Shallow sands supergroup, Sandy duplexes supergroup, shallow & Shallow loams supergroup	LS	Loam_Sand
Shallow sands supergroup, Shallow loams supergroup & Sandy duplexes supergroup, shallow	LS	Loam_Sand
Wet or waterlogged soils supergroup	W	Waterlogged
Wet or waterlogged soils supergroup & Ironstone gravelly soils supergroup	GW	Gravel_Waterlogged
Wet or waterlogged soils supergroup & Non-cracking clays supergroup	NCCCW	Clay_Waterlogged
Wet or waterlogged soils supergroup & Sandy duplexes supergroup, deep	SW	Sand_Waterlogged
Wet or waterlogged soils supergroup, Loamy earths supergroup & Non-cracking clays supergroup	NCCCLW	Clay_Loam_Sand_Waterlogged
Wet or waterlogged soils supergroup, Sandy duplexes supergroup, deep & Deep sands supergroup	SW	Sand_Waterlogged
Wet or waterlogged soils supergroup, Sandy duplexes supergroup, deep & Sandy duplexes supergroup, shallow	SW	Sand_Waterlogged
Wet or waterlogged soils supergroup, Sandy duplexes supergroup, shallow & Sandy duplexes supergroup, deep	SW	Sand_Waterlogged

APPENDIX D DYNAMIC SEGMENTATION TOOL

D.1 The dynamic segmentation tool

The following provides MRWA with a way to do a quick scan of any road to look for potential rehabilitation or pavement repair by combining the dynamic segmentation with Tableau/Power BI for visualisation. The thinking behind this is to address MRWA needs to identify 'pavement repair' prior to resealing, maximising the advantage of a very fine 10 m interval of TSD data.

D.1.1 Tolerance method

The tolerance method of dynamic segmentation is focused on combining contiguous segments of road into single segments. The process involves comparing sequential condition values to the running average of the current segment. If this value is within the predetermined tolerance range the segments are merged together and the next value is then compared. This continues until a value falls outside the tolerance range, at which point a new segment commences. The tolerance is a numerical value (user input) and the range is plus or minus. An example of this method is below:

Table D 2: Example of an output of the dynamic segmentation tool

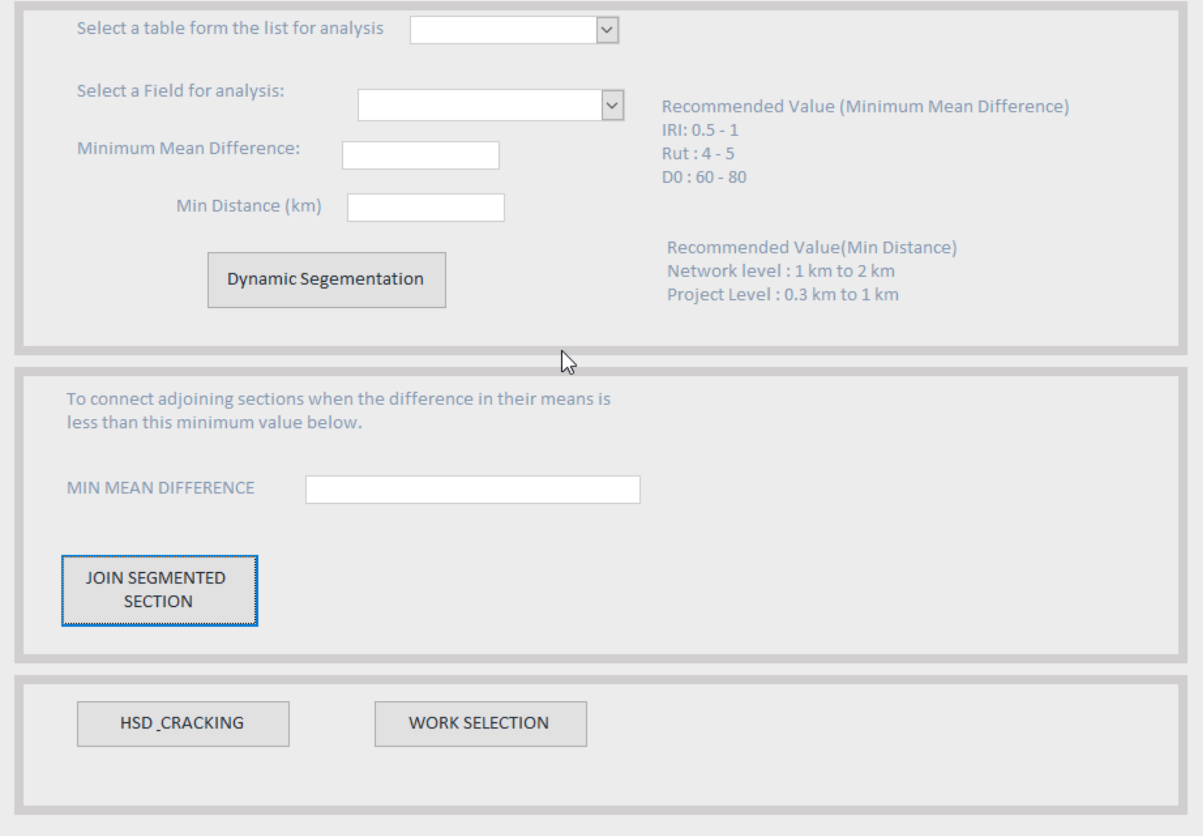
ROAD_ID	CHAINAGE	D0	SEGMENT AVERAGE	MAX TOL	MIN TOL	WITHIN TOL	SEGMENT
H003L_10	0.01	–261	–261			YES	1
H003L_10	0.03	–287	–274	–321	–201	YES	1
H003L_10	0.05	–258	–269	–334	–214	YES	1
H003L_10	0.07	–277	–271	–329	–209	YES	1
H003L_10	0.09	–249	–267	–331	–211	YES	1
H003L_10	0.11	–350	–350	–327	–207	NO	2
H003L_10	0.13	–291	–282	–410	–290	YES	2
H003L_10	0.15	–305	–285	–342	–222	YES	2
H003L_10	0.17	–306	–287	–345	–225	YES	2
H003L_10	0.19	–347	–293	–347	–227	YES	2

D.1.2 The tool

The dynamic segmentation process has been automated by using MS access. Segmentation is based on deflection (D_0) values from TSD deflection. This tool also can be used to segment the road section by other pavement performance indicators (e.g. roughness, rutting, texture and cracking) using the first two dropdown options as shown in the figure below. The Minimum-Mean-Difference text box let user specify the tolerance level for segmentation. The minimum distance text box allows user to specify the minimum segment length. After specifying all the required parameters, the Dynamic Segmentation button generates segments and creates new table, which later can be used to aid maintenance or rehabilitation programming.

The user may further combine contiguous segments of road into a single segment. The process compares sequential values to the current segment. If this value is within the tolerance range (user input) the segments are merged together, and the next value is then compared. The user can run this process multiple times.

Figure D 1: The Dynamic Segmentation tool User Interface



After segmenting the road based on deflection (D_0), the HSD_CRACKING button can be used to average other road performance measures (roughness, rutting, texture and cracking) into one table. This table can later be used to aid work selection. This table contains basic road information such as LINK_ID, Dist_from, Dist_to, etc. and thus can potentially be linked directly with MRWA pavement management system output.