Review of the NZ Transport Agency treatment selection algorithm September 2016

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An important note for the reader

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Contents

Executive summary 7
Abstract 10
1 Introduction 11
1.1 Background 11
1.2 Purpose of the TSA 11
1.3 Where does the TSA fit into the asset management process? 12
1.4 Summary of the TSA's strengths and weaknesses 12
2 Literature review 14
2.1 Treatment selection algorithms 14
2.1.1 TSA in New Zealand 15
2.1.2 Technical recommendations for highways no.12 (TRH 12) in South Africa 18
2.1.3 MODCALC in the Netherlands 18
2.1.4 AASHTO (1993) *Guide for design of pavement structures* 19
2.1.5 Asphalt PASER 20
2.1.6 Algorithm in Slovenia 21
2.2 Pavement maintenance/rehabilitation 22
2.2.1 Pavement life 22
2.2.2 Pavement behaviour and failure modes 23
2.2.3 Pavement rehabilitation practices in New Zealand 25
2.2.4 Prioritisation 25
2.2.5 Optimisation 26
2.3 Summary of literature review findings 28
3 Treatment selection algorithm process 29
3.1 The current process 29
3.1.1 Step 1: Compute area treatment costs 29
3.1.2 Step 2: Assess the need for resurfacing 29
3.1.3 Step 3: Estimate resurfacing cycle times 29
3.1.4 Step 4: Compute the present value of future maintenance 30
3.1.5 Step 5: Selection of shape correction treatment (SCT) option strengthening and smoothing 30
3.1.6 Step 6: Assess the need for resurfacing 30
3.1.7 Step 7: Resurfacing priority indicator 31
3.1.8 Step 8: Seal widening need 31
3.1.9 Step 9: Drainage maintenance needs 31
3.2 Strengths and weaknesses of the current system 31
3.3 Industry feedback 32
4 Use of new data sources 34
4.1 Introduction of new data sources into the TSA 34
4.2 Use of FWD data 34
4.2.1 Data parameters 35
4.2.2 Data processing 36

4.2.3 Data analysis ... 37
4.2.4 Correlations between FWD parameters and SIs ... 38
4.3 Use of HSD data for treatment option selection ... 42
4.4 Where no FWD data or HSD available ... 44
5 Improvements ... 45
5.1 Step 1: Compute area treatment costs ... 45
5.1.1 Calculation of general maintenance costs ... 45
5.1.2 Calculation of area treatment costs ... 46
5.1.3 Calculation of drainage treatment costs ... 48
5.2 Step 2: Assess the need for resurfacing ... 48
5.2.1 Current TSA assessment criteria ... 48
5.2.2 Accounting for maintenance costs ... 49
5.2.3 Accounting for thin asphaltic surfacings ... 50
5.2.4 Introduction of a composite index ... 51
5.2.5 Adding a year 3 selection criteria ... 51
5.2.6 Accounting for road classification ... 52
5.3 New step 2a: What SCT treatment is required? ... 53
5.3.1 SCT selection ... 53
5.4 Step 3: Estimate resurface cycle times ... 54
5.4.1 Current process ... 54
5.4.2 Accounting for maintenance works ... 55
5.5 Step 4: Compute the present value of future maintenance ... 55
5.5.1 Present TSA process ... 55
5.5.2 Review of the current calculation ... 55
5.6 Step 5: Selection of SCT options ... 58
5.6.1 Current TSA process ... 58
5.6.2 Recommended improvement ... 58
5.7 Step 6: Assess the need for resurfacing ... 58
5.7.1 Current TSA process ... 58
5.8 Step 7: Resurfacing priority indicator ... 59
5.8.1 Current TSA process ... 59
5.8.2 Recommended improvement ... 59
5.9 Step 8: Seal widening need ... 60
5.10 Step 9: Drainage maintenance needs ... 60
5.11 Additional comments ... 60
6 Conclusion and recommendations ... 61
6.1 Conclusion ... 61
6.2 Recommendations ... 61
6.3 Further work ... 62
7 References ... 63
Appendix A: Glossary ... 66

Executive summary

The objective of this research was to improve the treatment selection algorithm (TSA). TSA is used to forecast the timing and treatment required to maintain roads in good condition for the least whole-of-life cost in the short to medium term. The output was a list of candidate sites intended for validation in the field.

Since the TSA was developed in the 1980s:

- there has been considerable practical use of the current algorithm and a considerable programme of works completed and condition data collected each year
- the long-term pavement performance monitoring sites have yielded much practical information
- pavement and surface condition measurement techniques and parameters have developed
- economic analysis parameters have changed.

The algorithm, used to guide future surface and pavement works, needs to reflect current knowledge and recent experience. The TSA has performed well and a measure of its performance is reflected in the desire to update the algorithm. However, there have been a number of lessons learned in that time. Past assumptions, for example the progression of maintenance needs, need to be reviewed and replaced with evidence-based models. The greater use of thin asphaltic surfacings needs to be reflected in the TSA. The vehicle operating cost model and benefit-cost ratio funding mechanisms have been superseded. Learnings on pavement and surfacing performance from high-speed data capture and falling weight deflectometer (FWD) need to be incorporated. The quantity and accuracy of maintenance cost data is now much more prevalent, particularly with the use of RAMM Contractor.

Please note: Since the research to update the TSA began, development of a New Zealand road network classification system has been undertaken by the Road Efficiency Group. The road classification system has served to refocus maintenance investment into a level of service driven regime. In this light, while this research still stands, further analysis and consideration will need to be given as to its exact fit within the road classification system. Therefore when reading this document, the reader should do so with this caveat in mind.

Literature review findings

The New Zealand TSA compares very well internationally, especially when one considers it was developed over 25 years ago. South Africa has a simple condition-based trigger system to identify renewal candidates. MODCALC is a modular arithmetic calculator software package which has been set up to use FWD data solely for determining pavement or resurfacing need in asphaltic pavements. The American Association of the State Highway and Transportation Officials uses an empirical method while the asphalt pavement surfacing evaluation and rating algorithm, again an asphaltic pavement tool, is a trigger-based response to condition only. Slovenia has a similar flow chart system utilising a condition based algorithm incorporating FWD data.

One of the advantages of the TSA is the consideration of future surface lifecycles and a maintenance cost model in the economic assessment of treatment options, typically only seen in more complex predictive type systems. The trigger-based condition flowchart used to determine renewal need is sophisticated and based on a number of parameters. It calculates treatment and maintenance costs and examines drainage and seal widening. It allows a variety of economic scenarios to be tested. The TSA does not predict pavement or surface condition and looks at the short term only. However its purpose is to identify candidate sites for the short-term forward work programme, rather than quantifying medium to long-term needs and such a predictive function is therefore unnecessary.

Its main weakness compared with international practice is the lack of FWD information as an indicator of pavement strength and durability. The use of the benefit-cost ratio as an economic assessment is outdated now but is still more advanced than international practice elsewhere.

Conclusions

- Improvement recommendations focus on improving aspects of the algorithm rather than changing the core process itself. The most significant recommendation is the replacement of the benefit-cost ratio and vehicle operating cost-based benefit with a present value based. This also brings in a more flexible approach to the use of discount factors. The second major recommendation is to include FWD data, in particular, to determine the cause of pavement failure and therefore treatment type. It should be noted, however, that the treatment types recommended in this report are not much different from the current ‘smoothing’ ‘and strengthening’ options in terms of cost and assumed treatment form.

Summary of recommendations

- A name change could clear some of the misconceptions and unrealistic expectations of the TSA outputs received in industry feedback. A suggestion is the ‘candidate selection algorithm’. This, however, may need to be tempered with the familiarity many have with the name ‘treatment selection algorithm’. Changing the name may lose the link for practitioners trying to find the tool to use as well as losing the good reputation that the TSA has built up. This will need to be thought through carefully and the recommendation is only for consideration of a change.
- Include historic maintenance costs for testing in addition to current condition as maintenance interventions may mask indicators that the surfacing has reached the end if its life. The previous year’s maintenance costs plus recorded patches should be used to undertake this test. It would apply only to the pavement and surfacing activity classes.
- Add a new cost set table for thin asphaltic surfacings as their routine maintenance costs are different from those for chipseal surfacings. This can be done by splitting the TSA pavement type thin surfaced flexible cost set table into two parts, one for chipseal and one for asphaltic surfacings. The asphaltic surfacing cost set would need to be developed on a basis of repairs for relevant faults. The asphalt option would operate on a ‘like for like’ basis. The decision factors for TSA treatment types ‘reseal in budget’ and ‘reseal next time’ should be based on the TSA pavement type structural asphaltic concrete condition criteria for a resurfacing in budget or resurfacing next time, rather than on chipseal criteria. Testing would be needed to validate any values introduced into the TSA.
- Replace smoothing and strengthening options with a modified basecourse treatment or full pavement renewal. The treatment type options within TSA for smoothing and strengthening could be applied to the basecourse improvement or full pavement renewal respectively.
- Use traffic as a trigger for changing from a granular or stabilised base to a full structural asphalt construction, for example greater than 20,000 vehicles per day and/or quantity of heavy vehicles. This

would be user defined to allow networks to better align expenditure forecasts from the TSA to decisions that would have to be made in the field.

- Retain the two-year assessment window as without any forecasting of condition, three-year criteria are difficult to achieve with any credibility.
- The current TSA process has a single trigger level independent of road class. It would seem beneficial for the TSA process to allow for customisation according to road classification, or at the very least traffic volume. A simple method would be a user defined table, similar to the unit cost sets which are populated with standard default settings for each road classification. The values could either be user defined or hard coded as are the current TSA triggers. Any particular values would need to be tested
- Ensure a pavement renewal treatment will only be triggered if the treatment length meets the criteria for triggering a resurfacing, as described in the process in section 5.2.
- Where FWD data is available, use a combination of radius of curvature and central deflection to determine the failure mode and therefore treatment option for pavement renewal.
- Where high-speed data only is available, use a flushing test to determine whether a surfacing or pavement failure is the most likely failure mechanism.

  Where no such data is available, the following test is applied:

  - urban locations
    - <10,000vpd and/or collector or below basecourse improvement
    - >10,000vpd and/or arterial or above full pavement improvement
  - rural locations
    - basecourse improvement.
- Include a more definitive test for seal layer instability in the TSA, as this is a failure mechanism that is becoming more prevalent.
- Retain the current mechanism for calculating present value of future maintenance as the logic is strong and the programming in place already within the algorithm to perform the calculations.
- Enable the user to select the appropriate discount factor. This will make it easier in the future to reflect changes in the discount rate policy should the NZ Transport Agency have a shift in policy on this matter.
- Discontinue the benefit-cost ratio determination as the vehicle operating cost and benefit-cost ratio methodologies no longer match NZ Transport Agency policies and processes. Use the present value method to assess whether to select the shape correction treatment option.
- Even if resurfacing is the selected option, evaluate the treatment length for extreme levels of distress that would indicate a pavement renewal is still required.

Abstract

The objective of this research, carried out between 2012 and 2015, was to improve the treatment selection algorithm (TSA). The TSA is used to forecast the timing and treatment type of works required to maintain roads in good condition for the least whole-of-life cost in the short to medium term. The output was a candidate list of sites intended for validation in the field combined with recommended drainage improvements and funding estimates.

Since the TSA was developed, the long-term pavement performance monitoring sites have yielded much practical information; pavement and surface condition measurement techniques and parameters have developed; and economic analysis parameters have changed.

The algorithm, used to guide future surface and pavement works, needs to be updated to reflect current knowledge and recent experience. Recommended improvements include the consideration of thin asphaltic surfacings and maintenance cost data. The vehicle operating cost model and benefit-cost ratio funding mechanisms have been superseded and a new present value model is recommended. This incorporates new data sources now available such as falling weight deflectometer and high-speed data capture.

1 Introduction

1.1 Background

The objective of this research was to improve the treatment selection algorithm (TSA). The TSA is used to forecast the timing and treatment type of works required to maintain roads in good condition for the least whole-of-life cost in the short to medium term. The output was a list of candidate sites intended for validation in the field.

The treatment selection algorithm is now quite dated. Since it was developed:

- there has been considerable practical use of the current algorithm and a considerable programme of works completed and condition data collected each year
- the long-term pavement performance monitoring sites have yielded much practical information
- pavement and surface condition measurement techniques and parameters have developed
- economic analysis parameters have changed.

The algorithm, used to guide future surface and pavement works, needs to reflect current knowledge and recent experience. The TSA has performed well and a measure of its performance is reflected in the desire to update the algorithm. However, there have been a number of lessons learned in that time. Past assumptions, for example the progression of maintenance requirements, need to be reviewed and replaced with evidence-based models. The greater use of thin asphaltic surfacings needs to be reflected in the TSA. The vehicle operating cost (VOC) model and benefit-cost ratio (BCR) funding mechanisms have been superseded. Learnings on pavement and surfacing performance from high-speed data (HSD) capture and falling weight deflectometer (FWD) data need to be incorporated. The quantity and accuracy of maintenance cost data is now much more prevalent, particularly with the use of RAMM Contractor.

Please note: Since the research to update the TSA began, development of a New Zealand road network classification system has been undertaken by the Road Efficiency Group. The road classification system has served to refocus maintenance investment into a level of service driven regime. In this light, while this research still stands, further analysis and consideration will need to be given as to its exact fit within the road classification system. Therefore when reading this document, the reader should do so with this caveat in mind.

1.2 Purpose of the TSA

The TSA is a simple and easy to use tool that meets a number of needs for the road asset manager:

- At a treatment length level, it identifies a candidate list of renewal sites that can be taken for field validation to confirm the year 1 and 2 forward work programmes (FWPs).
- At a network level it serves to identify network pavement and surfacing renewal needs for a two-year period.
- At a network level it provides a prioritised list based on first year rate of return, which can be used as a basis for managing limited funding.

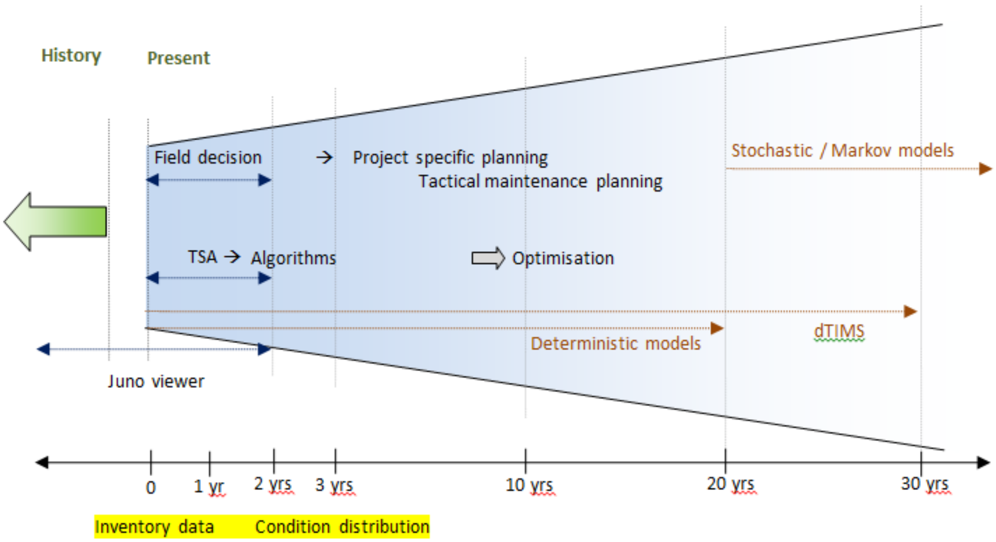
- At a network level, trend analysis from previous TSA runs can give an indication of whether FWP levels are sustainable or well targeted.
- At a national level it provides a useful tool for comparing network needs and benchmarking across different road networks.

The TSA is a very simple tool to run but behind the interface sits a complex but logical analysis to achieve the objectives outlined above. It is still very advanced in international terms in its thinking and approach despite its age. While it is overdue for an update, the fundamental principles and workings within it have been tested by the industry and its popularity is a support to its ongoing usefulness.

1.3 Where does the TSA fit into the asset management process?

The TSA is primarily used as a project decision tool for road maintenance planning. Figure 1.1 shows the TSA as a short-term maintenance planning tool, whereas optimisation and deterioration modelling would be required for tactical and strategic planning.

Figure 1.1 Road maintenance planning in New Zealand



1.4 Summary of the TSA's strengths and weaknesses

The TSA process has some very good strengths but some weaknesses as well.

Key strengths:

- Selection of resurfacing treatment is easily understood and uses a similar logic to most roading practitioners when creating FWPs during field inspection.
- Immediate general maintenance cost based on current condition is easily understood and reasonably robust.
- Capital cost calculation is easily understood and reasonably robust.

- Future general maintenance cost calculations are quite simplistic, outcomes are not sophisticated, and the literature research will support the fact that it is still more realistic and developed than other systems internationally.

Key weaknesses

- Using VOC and BCR with total transportation costs is no longer in vogue.
- It does not take account of past pavement maintenance costs and therefore identify where these have masked faults.
- It only looks forward by two years in terms of selecting a candidate programme list.
- It does not take into account FWD data used to estimate pavement strength or the number of equivalent standard axles (ESAs) for pavement loading.
- It does not adequately predict future treatments driven by multiple seals, layer instability, etc.
- There is poor consideration of thin asphaltic surfacings on flexible granular bases.
- It does not allow the user to change the triggers for selecting treatments (other than BCR) and therefore does not allow for variable levels of service. (It should be noted that feedback suggests this is a weakness. This report looks at the advantages and disadvantages of whether this should be an added feature. Adding it allows flexibility but also reduces the standardisation and comparison that a more fixed system allows.)

2 Literature review

2.1 Treatment selection algorithms

A TSA can be defined as a process which 'aims at obtaining results that mirror what engineering sense would dictate'. (The University of Birmingham and Transit NZ 2002). It is a system where defined values for defects in pavement condition trigger the need for a treatment option for a treatment length. Hence pavement condition data is the essential element of pavement management processes and is evaluated through measurement, observation and engineering judgement. Treatment selection processes are most often incorporated into computer-based programming systems (Robinson and Thagesen 2004) and prioritise sites for maintenance treatment for a failed pavement or for programming a maintenance strategy for a road network according to pavement condition (The University of Birmingham and Transit NZ 2002). Treatment selection processes may suggest several treatment options for a treatment length associated with different costs and treatment lives and it is through this system that engineers can decide on the next appropriate step in pavement treatment needs, complemented by their local knowledge and engineering judgement (Robinson and Thagesen 2004). It is most likely there will be an insufficient budget to fund all the treatments desired hence the need for prioritisiation and selection of the most appropriate treatment option.

Table 2.1 highlights a selection of different algorithms used worldwide and the parameters used as deciding factors in pavement rehabilitation.

Table 2.1 Comparison of different treatment selection algorithms used worldwide

Algorithm	Country	Data parameters	Fundamental principles
TSA (treatment selection algorithm)	New Zealand	• Shoving • Cracking • Scabbing • Potholes and patches • Flushing • Traffic volume • Rutting • Roughness • Texture • Surfacing material	Forward works programme
TRH 12 (Technical recommendations for highways) (CSRA 1997(	South Africa	• Cracking • Deformation • Patching • Ravelling • Smoothing • Riding quality • Rut depth • Skid resistance • Deflection data	Forward works programme

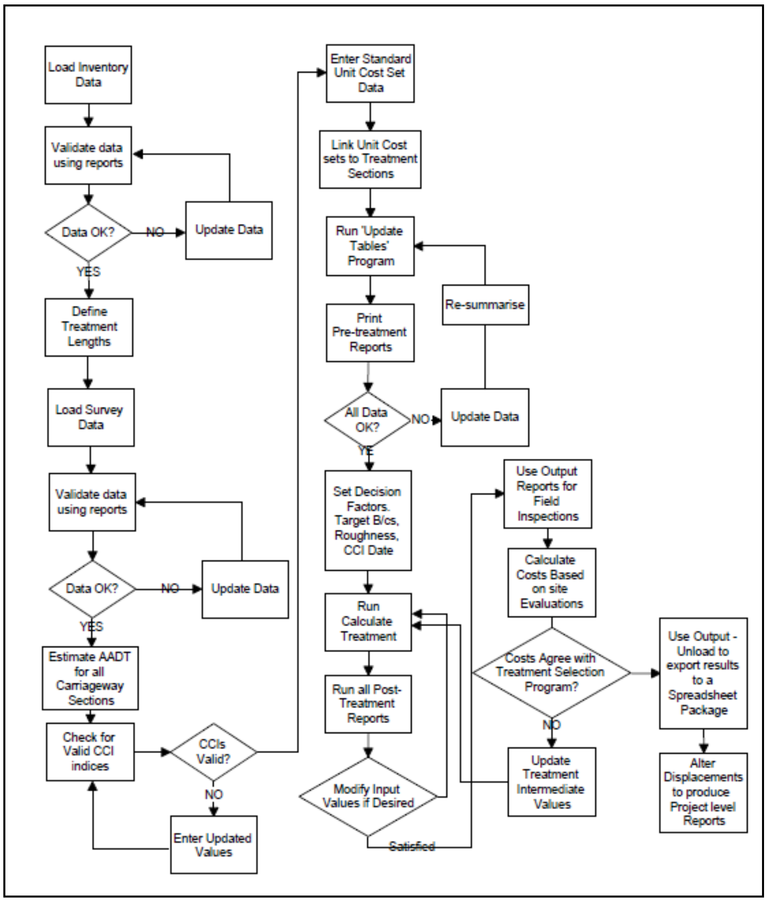
Algorithm	Country	Data parameters	Fundamental principles
MODCALC	Netherlands	• Pavement thickness • Deflection data • Traffic • Pavement temperature	Forward works programme
AASHTO 1993 method	USA	• Deflection data (FWD) • Traffic	Forward works programme
Asphalt pavement surfacing evaluation and rating algorithm (PASER)	USA	• Cracking • Rutting	Visual distress
Slovenian algorithm	Slovenia	• Rutting • Skid resistance • Condition of pavement surface • Roughness • Bearing capacity	Traffic safety

2.1.1 TSA in New Zealand

The TSA was introduced into Road Assessment and Maintenance Management (RAMM) in 1988 and last improved in 1997. It is a simple maintenance programme development tool within RAMM (Henning et al 2000) which formulates the optimal forward work as contained in an asset management plan (AASHTO 2011). Calibrated to local road networks it analyses the road network data for each treatment length hence giving a recommendation on the treatment needs in both the current and second year of the programme for a treatment selection. This is supplemented with an indicative benefit cost for rehabilitation works and the first year rate of return for reseals (Austroads 2002). The algorithm provides both for maintenance and rehabilitation works.

The TSA is more focused on thin surfaced flexible pavements (NZIHT 1999) with some consideration being given to structural asphalt. This accounts for the fact that the majority of New Zealand's sealed roads are thin surfaced materials. There are three types of data input needed from the user: pavement data sourced from RAMM, decision factors (including the allowable minimum BCR), target roughness value and seal life expectancy, and unit costs (for maintenance and rehabilitation). Roughness (reported in NAASRA counts per km) is a key trigger in determining maintenance treatments (Austroads 2002). The sequence of operations is shown in figure 2.1 (NZIHT 1999).

Figure 2.1 Sequence of operations flow chart

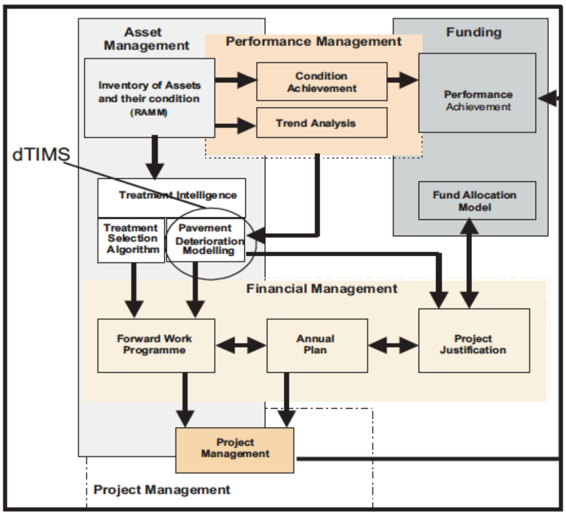


*CCI = Cost construction index

Source: NZIHT (2000)

Figure 2.2 highlights the typical interactions between components of the asset management process (Pradhan et al 2001). The outputs from the TSA, used in conjunction with a pavement management system such as the Deighton software product dTIMS, serve as an input into the long-term FWP process. The TSA is capable of predicting maintenance solutions for the short term which is useful in verifying treatments selected for that period (NZIHT 1999). The TSA does include the consideration of future surface lifecycles and a maintenance cost model in the economic assessment of treatment options. However it is not capable of predicting the pavement condition and maintenance needs in the medium to long term, or of testing those outputs in terms of economic benefits (Pradhan et al 2001).

Figure 2.2 Interactions of components within asset management

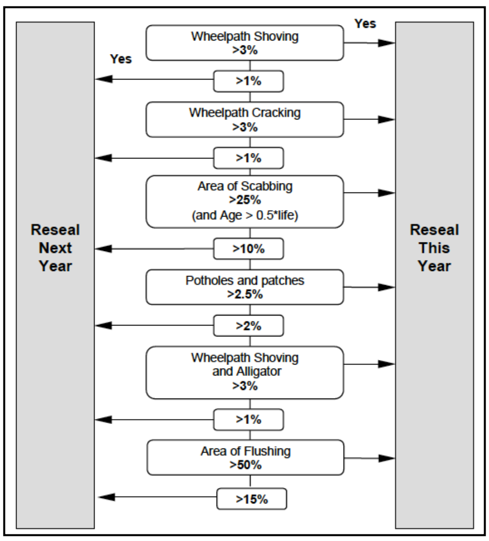


Pradhan et al (2001)

A TSA related to resurfacing needs is presented in figure 2.3 (Fawcett et al 2001). It utilises a decision tree and together with economic parameters based on empirical formulae, generates outputs (Henning et al 2000). TSA outputs consist of lists of treatment lengths, the recommended maintenance type for each, and the user benefits gained through maintenance (Gribble and Patrick 2008). As suggested, although the TSA shown would have been suitable for its intended use when it was first implemented, there are drawbacks in the logic. First, traffic volumes are not considered in any of the surface failure levels, and second, the interrelationship between the combination of surface failure types and the multiple distress modes are ignored.

Research notes that safety implications of maintenance treatments are not explicitly taken into account in the TSA and hence the recommendation is made that they should be included (Greenwood et al 1997). Maintenance alters the pavement surface and hence the accident rate.

Figure 2.3 Resurfacing selection logic in RAMM



Source: Fawcett et al (2001)

2.1.2 Technical recommendations for highways no.12 (TRH 12) in South Africa

The majority of road controlling authorities (RCAs) in South Africa use the network level PMS TRH 12 as guidelines for the management of pavement rehabilitation design (CSRA 1997). Table 2.1 highlights the condition parameters used as part of the initial assessment of pavement quality for rehabilitation. The guidelines consist of five chapters where sections two and three cover the condition assessment and rehabilitation design approach and options for pavements respectively. In condition assessment, there are various performance criteria which provide the engineer with the tools necessary for sound judgement.

Visually recorded distresses, such as cracking, deformation, patching, ravelling and smoothing, are measured as a percentage of a unit length and generally rated on a five-point scale, five being the most severe. Pavements are also categorised into four classes – A, B, C and D, and different performance criteria are assigned for each category. An example is illustrated in table 2.2 (CSRA 1997). Values falling outside the performance criteria can then be rated by the engineer on an urgency scale of one to five, ratings of four and five requiring immediate and urgent rehabilitation respectively. All pavements deemed in urgent need of a rehabilitation are further dealt with in section three of the guidelines for determining the type of rehabilitation required. It should be noted that this system, in contrast to the TSA, has varying levels of service by road classification or hierarchy.

Table 2.2 Road categories used for pavement design in South Africa

	ROAD CATEGORY			
	A	**B**	**C**	**D**
Description	Major interurban freeways and major rural roads	Interurban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Light pavement structure, rural access roads
Importance Service level	Very important Very high level of service	Important High level of service	Less important Moderate level of service	Less important Moderate to low level of service
TYPICAL PAVEMENT CHARACTERISTICS:				
RISK	**Very low**	**Low**	**Medium**	**High**
Approximate Design Reliability (%)	95	90	80	50
Total Equivalent Traffic Loading (E80/lane)*	3-100 x 10^6 over 20 years	0,3-10 x 10^6 Depending on design strategy	< 3 x 10^6 Depending on design strategy	< 1 x 10^6 Depending on design strategy
Typical Pavement Class**	ES10 - ES100	ES1 - ES10	ES0,003 - ES3	ES0,003 - ES1
Daily Traffic:: (e.v.u)***	> 4 000	600 - 10 000	< 600	< 500
Constructed Riding Quality: PSI*** HRI (mm/m or m/km) ***Severe (Terminal) Riding Quality*** PSI HRI (mm/m or m/km)	 3,5 - 4,5 1,5 - 1,0 2,5 2,4	 3,0 - 4,5 2,0 - 1,0 2,0 3,5	 2,5 - 3,5 2,7 - 1,5 1,8 3,9	 2,0 - 3,5 3,5 - 1,5 1,5 4,5
Warning Rut Level (mm) Severe (Terminal) Rut Level (mm)	10 20	10 20	10 20	10 20
Area/length of road exceeding the criteria set for the different road conditions (%)	5	10	20	50

Source: CSRA (1997)

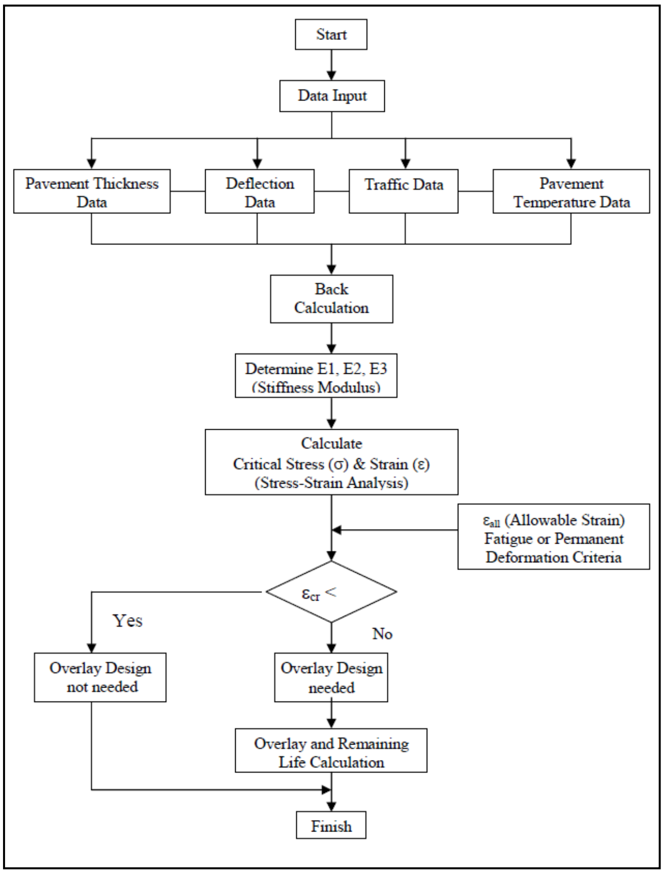
2.1.3 MODCALC in the Netherlands

MODCALC, a modular arithmetic calculator software package, has been set up to use FWD data solely for determining pavement or resurfacing need in asphaltic pavements. Information detailing the MODCALC programme is limited and the algorithm is shown in figure 2.4 (Subagio et al 2005). Deflections measured using FWD are entered into the programme where up to four structural layers can be specified. These are used to calculate layer moduli values from which critical stresses and strains are calculated.

Pavement thickness and a parameter not considered in the other algorithms in table 2.1 give the pavement temperature.

Critical stress and strain are calculated in the programme and are the deciding parameters for rehabilitation. If they fall within the permissible values no overlay is required.

Figure 2.4 MODCALC algorithm



Source: Subagio et al (2005)

2.1.4 AASHTO (1993) *Guide for design of pavement structures*

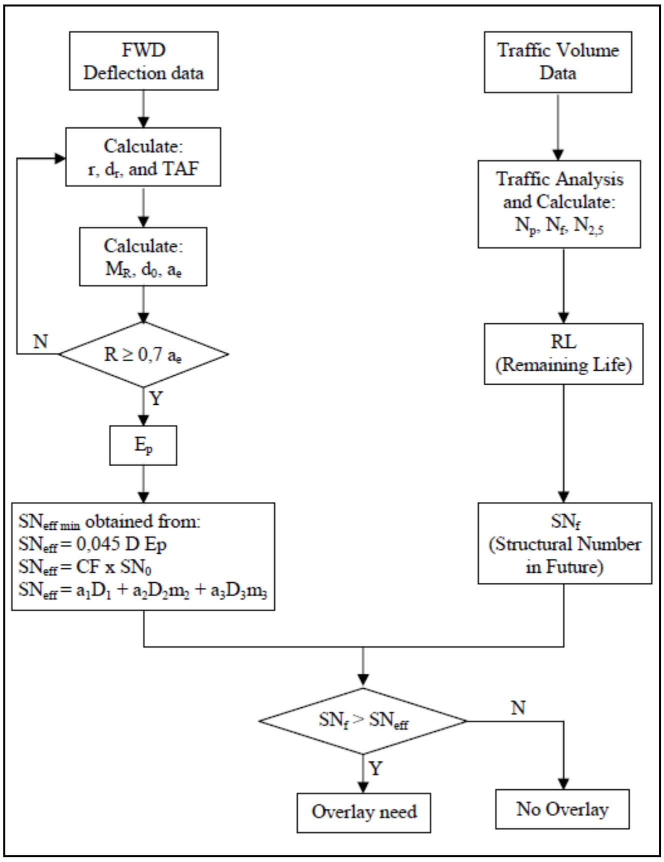
AASHTO (1993) is the primary pavement management tool used throughout the US and is recognised globally (Andersson 2007). It is an empirical method comprising a set of empirical equations and is essentially based on a value called the structural number (SN). The AASHTO algorithm is shown in figure 2.5 (Subagio et al 2005) and as can be seen, the algorithm relies on FWD data which ultimately results in the calculation of an effective structural number (SNeff). This represents the overall structural capacity of the pavement and is calculated for conventional asphaltic concrete pavements (Hoffman 2003) for each uniform pavement section (Zhao and Foxworthy 1999). The SN is derived through three methods dependent upon the data inputs available.

Other parameters required by the algorithm are the radius (r) or the distance from the centre of the FWD load plate, the deflection (D_r) measured at distance r. A temperature adjustment factor (TAF) is established based on the pavement thickness, air and pavement surface temperatures recorded during testing. E_p included in figure 2.5 represents the effective modulus of pavement layers above the subgrade and defines the overall stiffness of the pavement structure (Zhao and Foxworthy 1999). It is derived through back calculation.

The flow of calculations shown on the right side of figure 2.5 involves the derivation of the structural number required to carry future traffic (SN_f). As shown, when this parameter is greater than the calculated SNeff, a pavement overlay is required. This means that a pavement overlay is required based upon the stiffness and strength of the overall pavement.

The AASHTO algorithm is an empirical method although it is limited in terms of one location, one climate, limited traffic and one set of materials.

Figure 2.5 AASHTO algorithm

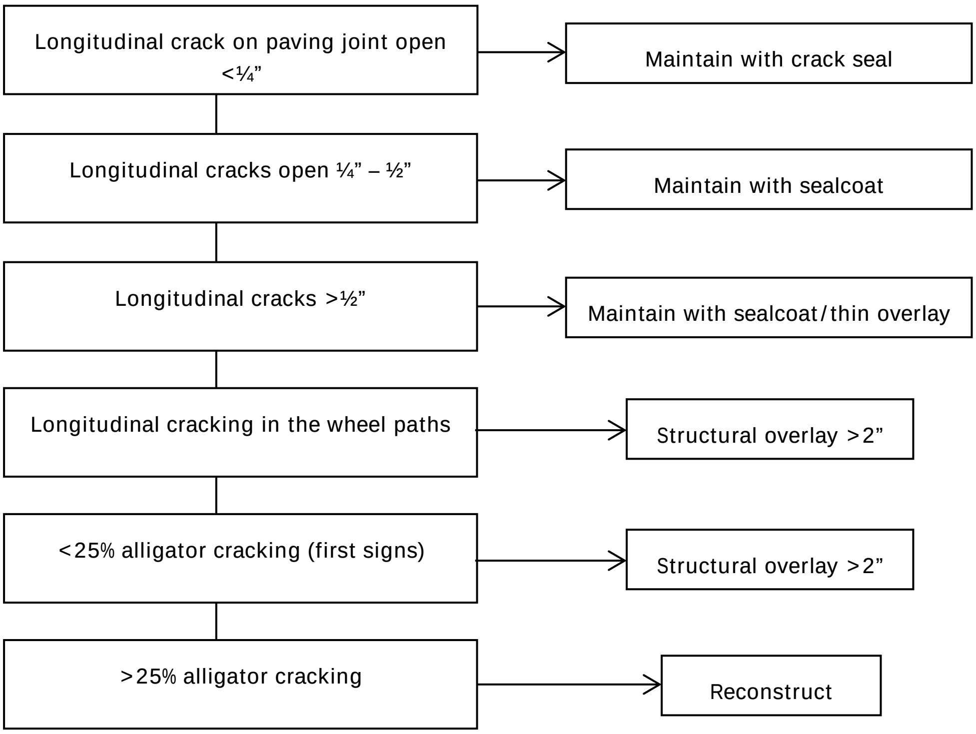


Source: Subagio et al (2005)

2.1.5 Asphalt PASER

The asphalt pavement surfacing evaluation and rating (PASER) algorithm is available for use by RCAs in the US (University of Wisconsin 2002). The logic of the algorithm is highlighted in figure 2.6. It is based upon the measurement of visual distress on the pavement surface. Parameters for rehabilitation include how open the longitudinal and transverse cracks are, rut depth, and percentage of the surface affected by block cracking. These are followed by recommended maintenance treatments. There is no definition outlining how the percentage of the surface affected by block cracking is measured.

Figure 2.6 Asphalt PASER algorithm



It should be noted that the surfacing cracking open more than a quarter inch would not be appropriate for the flexible base course pavements typical in New Zealand.

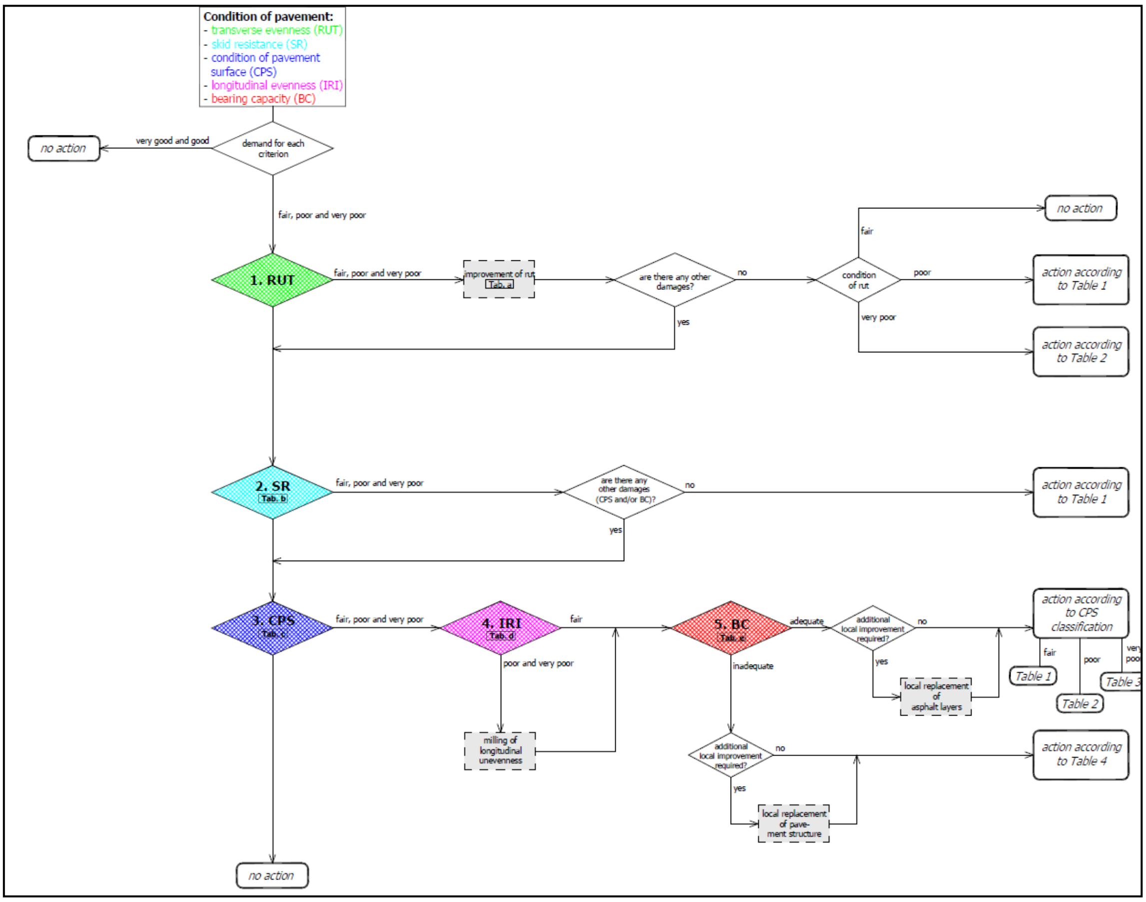
2.1.6 Algorithm in Slovenia

The Slovenian algorithm shown in figure 2.7 (Jamnik et al 2006) is based on traffic safety in terms of how the condition of the pavement surface affects the road user in riding quality. It is used for flexible pavements and suitable for all traffic loads. There are five condition parameters:

- rutting (RUR)
- skid resistance (SKID)
- condition of pavement surfacing (CPS)
- roughness (IRI)
- bearing capacity (BC).

If upon inspection at least one of them is applicable, rutting is checked first. Pavements are rated using one of five classes – very good, good, fair, poor and very poor, and there are referenced tables which define the range limits for each class. Pavements exhibiting a very good or good condition are not assessed further in terms of rehabilitation. The condition of pavement surface includes cracks, ravelling, potholes and patches, and the bearing capacity is measured using either FWD or a deflectograph. There are also reference tables which define the action that should be undertaken to fix the problem.

Figure 2.7 Slovenian algorithm



Source: Jamnik et al (2006)

2.2 Pavement maintenance/rehabilitation

2.2.1 Pavement life

Pavement design in New Zealand is based on Austroads (2005) *Pavement design: a guide to the structural design of road pavements*, which focuses on minimising the subgrade strain while taking into account the design traffic. Therefore New Zealand pavement design heavily relies upon the subgrade strain (Gribble and Patrick 2008).

The remaining life of a pavement is defined as the time in which a pavement is serviceable before reaching a terminal state (Daly 2004). This is based upon the elastic deflection of the pavement where a pavement exhibiting a high deflection will have a shorter remaining life compared to a pavement with a low deflection under the same traffic loading. Rutting of the subgrade is the factor related to deflection indicating the terminal condition of a pavement. However, from New Zealand experience, roughness and maintenance are more likely to determine when a pavement requires rehabilitation or other intervention to relieve distress.

Pavement life can also be viewed from an economic stance (Arampamoorthy and Patrick 2010) where if the net present value of future maintenance is greater than the cost of rehabilitation, then the pavement can be considered to have reached the end of its life. However, this analysis needs to fully explore the economic benefits of a range of treatment and renewal options to determine the optimum maintenance strategy.

Remaining life is based upon three main factors or influences – design loading, strength and environmental factors. Design loading is based on the number of ESAs that a pavement will bear over its design life. Generally granular pavements are constructed with a design life of 25 years. Although this methodology is reliable when considering a base loading, it does not take into account the increase in traffic loading caused by factors such as geometric stresses from vehicles travelling around tight corners and accelerating on gradients. Roughness and travel speed have significant impacts on traffic loading. It should also be noted that the design life is based on achieving a terminal state such as a rut depth of 20mm. It does not follow, however, that the achievement of the terminal state requires renewal of the pavement. Maintenance treatments will prolong pavement life beyond this.

For pavement strength, the elastic modulus is used as a measure where an elastic deflection is calculated at critical points within a pavement structure under a single ESA. Through use of an empirically derived transfer function the elastic deflection is converted into a plastic deformation. Pavement strength is affected over time through factors such as environmental effects and densification of pavement layers through traffic loading. Lastly, environmental loading takes into account the effect of water on pavements and aggregate breakdown. Although this factor is not currently included in the analyses of pavement life, it could be considered in terms of predicting pavement remaining life.

2.2.2 Pavement behaviour and failure modes

The behaviour of a flexible pavement under loading is a complex mechanism and is attributed to its multi-layered structure. It is determined by the pavement's response to surface stress from traffic. In definition it is a function of its original construction and the environment it is subjected to (Masad 2004), that is, stresses caused not only by traffic loading but environmental factors as well. The layers comprising the structure react differently to these stresses due to the variability in material strength and characteristics (Henning 2009). Pavement behaviour gives an indication of the pavement condition over time.

Pavement behaviour is linked to failure modes. The response of a flexible pavement to a stress indicates the source of failure, therefore exposing the cause and mechanism of distress (Masad and Little 2004). There are two distinct mechanisms that lead to pavement failure – functional and structural defects (El-Shafei 1986). Functional defects are broad in definition and involve the deterioration of the pavement's wearing surface. Vehicle drivers experience its effects as discomfort when driving over an irregular surface, or through the fatigue loading on vehicles, and these effects are largely determined by the surface roughness of the pavement.

Structural defects by comparison are more complicated as they involve the layers within the pavement structure. They are the consequence of the subgrade or other pavement layer failing to perform its function so the pavement structure can no longer sustain the impact of the loads it is subjected to. Each pavement layer has a distinct mode(s) of failure and the effect may migrate to the surface of the pavement causing defects that appear as soft or wet patches, larger depressions, or cause a loss of pavement (Henning 2009). As a whole, failure modes are described according to the location of the distress mode.

There are three main failure modes that occur as a structural defect: rutting, cracking and shoving. Rutting is the result of compressive strains on the subgrade (Salt et al 2010) and there are two basic types:

consolidation and instability rutting. Consolidation rutting is the result of either a repetition of traffic loads causing a compaction and hence a reduction in the air voids in the asphaltic concrete layer, or a permanent deformation of the base or subgrade (Drakos 2010). It results in longitudinal wheel paths along the surface and can be visually identified by depressions (Abdallah and Nazarian 2011). Instability rutting is a characteristic of the surface layer only. In an asphaltic concrete surface layer it is the result of an inadequacy in the hot mix asphalt design (Abdallah and Nazarian 2011). It results in depressions attributed exclusively to the asphalt layer and can be visually identified by the humps that form on the side of the ruts. It can also occur in multi-layer chipseal pavements.

Cracking occurs in the pavement and surfacing layers and there are four distinct types: alligator, block, longitudinal and transverse cracking (Henning 2009). Alligator cracking is a series of interconnecting cracks which are initiated in the wheel paths and progress out along the surface under repeated traffic loading. Its pattern resembles that of an alligator’s skin. Causes are related to an inadequacy in asphalt layer thickness, with excessive pavement deflection or binder lean asphalt resulting in traffic loads heavier than what the pavement can withstand (Abdallah and Nazarian 2011). Depending upon the severity it is an important mechanism of distress as its effects stem from pavement stiffness and strength.

Cracking allows for the infiltration of water into the underlying pavement layers, hence accelerating the rate of deterioration. Block cracking is a series of perpendicular cracks which section the road surface into rectangular blocks. These cracks are related to the age of the pavement where it shrinks and hardens over time due to temperature variations or a thin stiff over-cemented layer over a weak pavement layer. Alternatively over-stabilisation can cause shrinkage cracks as well. Longitudinal cracks are cracks that form parallel to the centre line of the pavement and do not occur in the wheel path. They are not associated with traffic loading and are associated with poor pavement construction or shrinkage at construction joints (Henning 2009). Transverse cracking forms across the lane or full width of the pavement. It occurs as a result of hardening as the pavement ages and is also due to shrinkage caused by variations in temperature (Henning 2009).

Shoving is the longitudinal surface displacement of the asphalt or surfacing (Abdallah and Nazarian 2011). Lateral displacement can also occur in high stress curves. It is similar to instability rutting in that the loading induces shear stresses that shove material away from the loaded area (Drakos 2010). It is typically caused by the acceleration and deceleration of vehicles and can also be the result of a poorly designed asphalt mixture (Abdallah and Nazarian 2011). A lack of cohesion between the hot mix asphalt and underlying layer or shear strength in granular layers also leads to shoving where the resistance to horizontal stresses is inadequate.

There are a number of factors that can cause roughness: traffic volume, load magnitude, pavement construction, pavement maintenance, pavement materials and environmental conditions. Traditionally New Zealand uses the National Association of Australian State Road Authorities (NAASRA) measurement to indicate roughness; however, this is gradually being replaced by the international roughness index (IRI).

Rutting and roughness are frequent drivers for pavement rehabilitation. Austroads (2000) defines a pavement terminal condition by recommending values for both rutting and roughness. However, it is not clear whether both parameters need to exist for the pavement to have reached the end of its life. Often rutting is associated with low roughness readings. Austroads suggests an average rut depth of 20mm and roughness of approximately three times the initial roughness.

Pavement design methodologies used in South Africa are similar to those used in Australia and New Zealand. They determine the pavement life for each pavement layer (including the subgrade) by combining a linear elastic analysis with performance models for each layer found empirically. The integration of South African design methodologies into the New Zealand methodology is worth further investigation.

2.2.3 Pavement rehabilitation practices in New Zealand

Pavement rehabilitation is a structural or functional enhancement of a pavement to improve or restore its performance in terms of ride quality, waterproofness, integrity and strength (Austroads 2000). It is distinguished from pavement maintenance activities which involve preserving the pavement condition in order to achieve its design life. An important aspect of rehabilitation is identifying what the deficiencies and needs are. This is gained through analysing data from the pavement history available from RAMM. Visual assessments and automated surveys such as HSD capture identify the current condition of the pavement. Rehabilitation options can then be formulated, an economic analysis made of the options, and the most feasible option chosen.

Rehabilitation options are used according to where in the pavement structure the failure occurs. New Zealand employs a mechanistic procedure for rehabilitation.

2.2.4 Prioritisation

2.2.4.1 Economic analysis

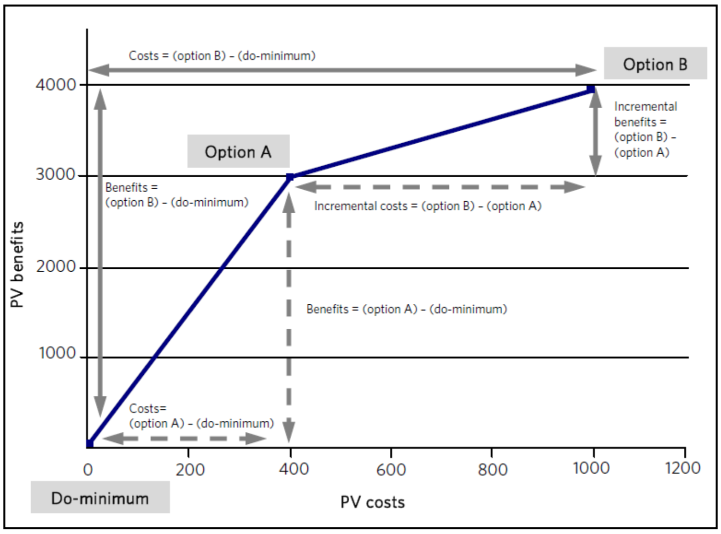
Economic evaluation and preparation for funding of all transport activities including pavement design, rehabilitation and maintenance works, are performed in accordance with the Transport Agency's *Economic evaluation manual* (EEM) (NZ Transport Agency 2013). All RCAs in New Zealand must use this economic analysis procedure in justifying their pavement renewals. Prior to 2013, the EEM volume one used benefit-cost calculations in the prioritisation of pavement renewals, hence the inclusion in the TSA. Currently it prescribes a PV analysis and excludes VOC benefits.

When undertaking roading activities the options should be economical in terms of initial construction costs and recurring maintenance costs. The purpose of economic analysis is to rank the feasible options of an activity according to the benefits and costs. PV and benefit-cost calculations are performed where a discount rate is applied to discount all future benefits and costs of activities to their net PV (Li et al 2011), usually over a design period of 30 years.

The discount rate currently used in New Zealand is 6% and is a key factor in economic analysis. The discount factor used in the TSA is 10%, the discount factor at the time of coding. Benefits include the reduction in VOC resulting from a decrease in roughness, whereas costs can also include maintenance activities. Benefits and costs are compared by means of a BCR as defined in the EEM as 'the PV of net benefits divided by the PV of net costs'. For more complex systems involving the comparison of different options an incremental benefit-cost analysis is performed. The incremental BCR compares the benefits of different options against each other.

An example of this is shown in figure 2.8 (NZ Transport Agency 2010) where options A and B are more beneficial than the do-minimum option but also cost more.

Figure 2.8 Example of an incremental BCR



Source: NZ Transport Agency (2010)

Resurfacing is often the most cost-effective option in terms of renewal maintenance. However there is a limit to the number of reseals that can be made before it becomes infeasible and rehabilitation of the pavement is the only option. Although economics have a high influence in terms of whether a pavement is rehabilitated, it is also suggested that if a pavement surface begins to show defects above the suggested limit, they should be corrected to maintain the minimum level of service required.

2.2.5 Optimisation

To achieve a balance between maintenance needs and budget constraints the process of project prioritisation should be combined with budget optimisation. Optimisation is the process used to make a design or decision-making system efficient by maximising productivity or minimising waste, specifically, achieving the 'best value' for the system based on a set of constraints. In the context of road asset management, the various maintenance options need to be analysed for their desired outcomes while taking into account constraints such as available budget or risk.

The use of the optimised maintenance project selection process ensures that when maintenance is carried out, it is targeting areas that would otherwise be the costliest if failure occurred, while at the same time ensuring optimum timing of the maintenance. This method of optimised maintenance planning, as opposed to site-specific treatments, achieves an improved network condition plus significant cost savings.

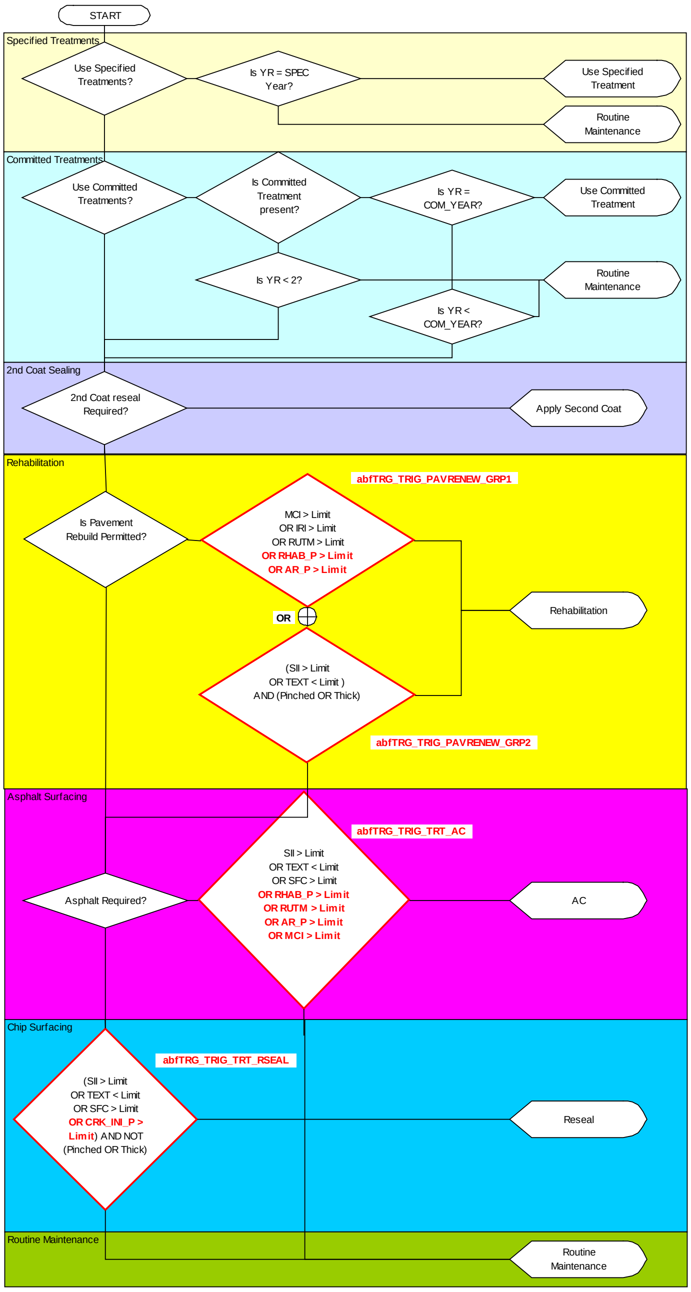
The NZ dTIMS system is used as a tool to conduct performance forecasts for selecting the sites that were most in need of maintenance. The ease with which the NZ dTIMS system could be aligned to each road network allowed it to be effective in the optimisation process.

In terms of the context of this research it is worth noting that the NZdTIMS system has three modes for the analysis of road networks:

- The optimised programme is used to determine the best investment strategy for a set of budget constraints. From this, the analysts would be able to determine the best split between reactive and proactive treatment plus the most likely projects involved with each scenario.

- The rule-based analysis is used during the initial set-up of the model to confirm the performance of a network. No funding constrains are applied to a decision algorithm similar to the TSA. The difference with the dTIMS rule-based analysis is that it works on the bases of forecasted data.
- The specified analysis takes an existing programme (say the 10-year FWP) and applies forecasting models to test the consequences of the given programme. The algorithm for the rules-based analysis is depicted in figure 2.9 (IDS 2009).

Figure 2.9 NZdTIMS rule- based analysis



Source: IDS (2009)

2.3 Summary of literature review findings

The New Zealand TSA compares very well internationally, especially when one considers it was developed over 25 years ago. South Africa has a simple condition-based trigger system to identify renewal candidates. MODCALC uses FWD solely to determine pavement or resurfacing need in asphaltic pavements. ASSHTO uses an empirical method, while PASER, again an asphaltic pavement tool, is a trigger-based response to condition only. Slovenia has a similar flow chart system utilising a condition-based algorithm incorporating FWD data.

One of the advantages of the TSA is the consideration of future surface lifecycles and maintenance costs in the economic assessment of treatment options, typically only seen in more complex predictive type systems. The trigger-based condition flowchart used to determine renewal need is sophisticated and based on a number of parameters. It calculates treatment and maintenance costs and examines drainage and seal widening. It allows a variety of economic scenarios to be tested. While TSA takes into account future surface lifecycles and utilises a maintenance cost model in the PV analysis of treatment options, it does not predict pavement or surface condition. Its purpose is to identify candidate sites for the short-term FWP, rather than quantifying medium to long-term needs and such a predictive function is therefore unnecessary.

Its main weakness compared with international practice is the lack of FWD information as an indicator of pavement strength and durability. The use of the BCR as an economic assessment is outdated now but is still more advanced than other methods used in international practice.

3 Treatment selection algorithm process

3.1 The current process

While coded within RAMM, the algorithm itself is not widely documented elsewhere. The two core reference documents available are:

- *RAMM treatment selection workshop manual,* version 3.7 (NZIHT 1999) which details the thinking and process behind the RAMM treatment selection algorithm. It covers most of the theory behind the algorithm and some of the detail, but it does not detail the algorithm itself.
- *Transfund NZ research report 87* 'Review of RAMM treatment selection process for state highways and local authority roads in New Zealand' (Beca Carter Hollings and Ferner Ltd 1997).

These documents form the basis for understanding and analysing the current state of the TSA algorithm, and for making recommendations for its improvement. They are important reference documents to be considered in conjunction with the recommendations of this report.

The current process can be summarised as follows (NZIHT 1999):

3.1.1 Step 1: Compute area treatment costs

Before undertaking any of the four possible treatment types listed below, the costs of necessary preliminary maintenance and drainage repairs are calculated.

The four treatments are:

- general maintenance
- resurfacing
- smoothing (smoothing overlay)
- strengthening (rehabilitation).

The preliminary repair costs are estimated from deficiencies revealed in the pavement condition rating.

3.1.2 Step 2: Assess the need for resurfacing

On the basis of the surface condition ratings, the need for resurfacing the pavement is assessed, assuming at this stage that shape correction is not an option. Pavements are assessed as requiring a resurface in the budget year, the year following or at a later date. If the seal does not appear to be in the first two categories, it is assumed it will probably last the normal life for that type of seal at that particular traffic level.

3.1.3 Step 3: Estimate resurfacing cycle times

The likely length of the resurfacing cycle is estimated following each of the four area treatment choices listed in step 1 above.

If the existing pavement distress is very high, a check is made to see if some of the distress could have been economically averted by shortening the resurfacing cycle by one or two years. If so the future resurfacing cycle is adjusted accordingly.

Generally the life of future surfacing is estimated from the performance of the current surfacing rather than simply expected life. In the case of premature failure of the current surfacing (less than 70% of the normal life span for the traffic level), the condition of the pavement is checked. If the drainage is seriously deficient, the assumption is made that drainage improvements will partially restore the pavement performance in the future.

The types of surface deficiencies are also checked by the TSA and if these are not pavement related then the pavement is assumed to be capable of supporting the future surfacing for a normal lifespan.

If the drainage is not deficient and the types of extensive distress do not indicate a design or construction fault, the pavement is assumed to be incapable of supporting future surfacing for normal life spans. Consequently the pavement will have a short resurfacing cycle and high maintenance costs, which will significantly increase the PV of future maintenance and resurfacing.

3.1.4 Step 4: Compute the present value of future maintenance

The discounted PV of future reseals and general maintenance activities is computed for each of the four area options.

The general maintenance costs are not estimated on any engineering or theoretical basis but are arbitrarily assumed to occur mainly in the years immediately before each resurfacing, building up to a peak in each resurfacing year. Hence the general maintenance PV is a function of the length of the resurfacing cycle only.

The discount rate used is 10%.

3.1.5 Step 5: Selection of shape correction treatment (SCT) option strengthening and smoothing

SCTs are assumed to provide a similar level of road roughness after treatment. Thus the option with the lowest total treatment cost plus discounted maintenance cost is the preferred SCT option. The expected values are set by the user.

3.1.6 Step 6: Assess the need for resurfacing

The preferred SCT option and the non-SCT alternative already decided in step 2 are compared.

If the total treatment cost plus discounted maintenance cost of the SCT option is less than that of the non-SCT alternative, the SCT option is automatically given a higher priority. In other cases the BCR of the SCT is computed.

Benefits are accrued from reduced roughness levels, which lower the VOC. If the BCR exceeds a user supplied cut-off value, the preferred SCT option is selected. The benefit–cost value is used as a priority indicator for the list of SCT treatments.

3.1.7 Step 7: Resurfacing priority indicator

If the non-SCT option is selected and this option is for a surfacing in the budget year, a resurfacing priority indicator is calculated.

The additional cost of maintaining the road in good condition for an additional year is estimated by assuming that most of the defects shown in the rating will require correction in the budget year and a proportion of these will recur and require correction before resurfacing the following year.

This ‘delay cost’ is divided by the cost of the resurfacing to give the first year rate of return which is used as a priority ranking indicator for the resurfacing list.

3.1.8 Step 8: Seal widening need

The need for seal widening (for maintenance reasons rather than safety) is considered. The annual rate of deterioration is calculated by dividing the amount of edge break plus edge break repairs by the surfacing age. If the rate exceeds 5%, the road is reported for a possible widening.

3.1.9 Step 9: Drainage maintenance needs

The drainage maintenance costs and requirements are listed. For resurfacing or SCT, all defects are assumed to require rectification.

3.2 Strengths and weaknesses of the current system

As described above, the current TSA process is complex and well thought out. It certainly compares well with any other similar international systems. It considers a breadth of parameters and accounts for a number of different internal tests and logic loops. Its strengths can be summarised as follows:

- Selection of resurfacing treatment uses a similar logic to most roading practitioners when creating FWP during field inspection. It should be noted that the current logic utilises a series of discrete parameters rather than a composite index. This is discussed in more detail in section 5.2.4.
- Immediate general maintenance cost based on current condition is easily understood and reasonably robust.
- Capital cost calculation is easily understood and reasonably robust.
- Future general maintenance cost calculation is simplistic but is only used to determine the PV for various treatments. It does, however, provide a consistent platform from which to compare options.

However, the current process has not been updated since 1997 and has a number of weaknesses:

- Using VOC and BCR with total transportation costs is no longer in vogue.
- It does not allow the user to change the triggers for selecting treatments (other than BCR) and therefore does not allow for variable levels of service. (It should be noted that feedback suggests this is a weakness. This report looks at the advantages and disadvantages of whether this should be an added feature. Adding it allows flexibility but also reduces the standardisation that a more fixed system allows.)

- It does not take account of past pavement maintenance costs and therefore identify where these pavements have masked faults. The use of pavement patches recorded in the rating process should also be included.
- It only looks forward by two years in terms of selecting a candidate programme list (need to decide where TSA finishes and pavement performance modelling starts).
- It does not take into account FWD data used to estimate pavement strength or the number of ESAs for pavement loading.
- It does not adequately predict future treatments driven by multiple seals, layer instability etc.
- It does not allow for a variable level of service for different road categories although it is not level of service focused.
- There is poor consideration of thin asphaltic surfacings.

In addition, the research brief identified that the following needed to be included:

- the context of sites such as urban or rural
- future traffic demand.

3.3 Industry feedback

At the 2013 RIMS Forum, feedback was sought on the current TSA. Common responses were:

- Useful for getting candidate sites for renewal
- Well used but not so much for setting budgets and drainage items
- It is a ‘black box’ with little understanding of the algorithm set up
- Poor quality data affects the results
- Needs to be differentiated from dTIMS
- Like to see a three-year window over the current two-year window
- Include crash data
- Include strength data
- Texture model requires rethink
- Treatment costs change depending on existing surfacing
- What surfacing lives are used and which are correct
- Would like to see ability for some customisation

There still seemed to be an expectation that the TSA would produce a FWP for the coming year and some disillusionment with the results. This understanding missed the purpose somewhat in that it is just a desktop analysis providing candidate sections for field validation. It is also a tool in the development of the FWP giving a guide to network demand, particularly with comparing trends in TSA recommendations over time. These expectations are very high and can lead to some disappointment and disillusion with the results. Therefore, some management of expectations should also be recommended to improve its use. It needs to be understood as a short-term tool for selecting candidate sites, which serves a different purpose from pavement deterioration modelling which looks to the medium to long term.

Finally, a name change could clear some of the misconceptions and unrealistic expectations of the TSA outputs received in industry feedback. A suggestion is the ‘candidate selection algorithm’. This, however, may need to be tempered with the familiarity many have with the name ‘treatment selection algorithm’. Changing the name may lose the link for practitioners trying to find the tool to use as well as losing the good reputation that the TSA has built up.

4 Use of new data sources

4.1 Introduction of new data sources into the TSA

Since the TSA was developed and updated, there have been significant changes in technology for data capture and testing. In particular, HSD has allowed automated network-wide surveys of texture and rutting to be collected in conjunction with roughness. FWD data is available giving much improved data around pavement strength and therefore performance of pavements and surfacings. It is proposed that where possible, simple tests giving guidance to the failure mechanism present should be used to determine the subsequent treatment option using these data sources. This test is dependent on the data available for each treatment length.

For example, based on the availability of the following data

- FWD data: Deflection and radius of curvature (RoC) calculations (Horak 2008) can be used to determine whether upper or lower layer failure is likely (see section 4.2.4). FWD data can also be used to assess when a less strain tolerant current surfacing such as asphalt is required to be placed over a more tolerant chipseal surfacing. The assessment is required to assess whether the current pavement and therefore whether some strengthening is required should.
- HSD: A failure mechanism based on flushing (Kodippily and Henning 2011) (see section 4.3).

4.2 Use of FWD data

For the FWD data, two analyses were undertaken. The first analysis determined the parameters defining critical failure, boundary values and introduced strength parameters as decision criteria for the TSA. The analysis compared base or unprocessed FWD data with strength indices and involved plotting SIs against their corresponding condition and FWD parameters as scatterplots. Parameters with correlations, as detailed in section 4.2.4, were then proposed for use in the improved TSA.

The second analysis tested predefined boundary condition values for FWD parameters to define the pavement condition. It involved plotting each group of significant FWD parameter into histograms. The distributions were then grouped into three categories according to their structural condition – sound, warning and severe. These categories were defined according to similar research conducted by Horak (2008, p4) and are reproduced in table 4.1.

Table 4.1 FWD parameter structural condition rating for granular base pavements

Structural condition rating	FWD parameters				
	D_0 (µm)	RoC (m)	BLI (µm)	MLI (µm)	LLI (µm)
Sound	<500	>100	<200	<100	<50
Warning	500–750	50–100	200–400	100–200	50–100
Severe	>750	<50	>400	>200	>100

D_0 = maximum deflection; RoC = radius of curvature; BLI = base layer index; LLI = lower layer index

Source: Horak (2008)

This criteria was then used in the following decision matrix to determine the failure mechanism.

Table 4.2 Decision matrix for determining pavement failure mechanism

RoC – severe	D0 – severe	D0 – warning	D0 – sound
RoC – severe	Pavement	Basecourse	Basecourse
RoC – warning	Pavement	Basecourse	OK
RoC – sound	Pavement	OK	OK

The data was analysed at the network level through the provision of RAMM road network data from the following RCAs – Manukau City Council (MCC), Taranaki Regional Council (TRC) and North Shore City Council (NSCC). Data from TRC was that of the state highway network as included in the region.

4.2.1 Data parameters

Three groups of parameters as shown in table 4.3 were of interest – condition, deflection bowl data (derived from FWD data) and structural indices (SIs).

As a replacement for adjusted structural number (SNP), Salt et al (2010) proposed an alternative structural parameter, termed structural index (SI). For each of the recognised structural distress modes (ie rutting, roughness, cracking and shear) a corresponding SI was developed. Salt et al (2010) investigated the process of refinement of partly derived SIs for rutting, roughness, cracking and shear. Each SI is mechanistically derived and has the same range (1 to 8) and general distribution as the traditional SNP.

Salt et al (2010) showed that the SIs could give guidance to the failure mechanism and therefore guidance to the likely treatment option. However, this work is not available with all FWD data. It would be preferable therefore to use base parameters available from FWD data if the correlation was strong enough to use in place of the SIs. The sections below detail the analysis and the results in section 4.2.4 confirm the correlations between FWD and SIs.

Table 4.3 Variables used in analysis

Data type	Parameter
Condition data	• Rutting • Roughness • Rut rate
Deflection basin parameters (FWD)	• D_0 • RoC • BLI • LLI
SI	• Structural index for rutting (SIrutting) • Structural index for roughness (SIroughness) • Structural index for flexure (SIflexure)

Three separate spreadsheets containing historical data extracted from RAMM were provided for each RCA, one spreadsheet for each group of parameters. The FWD data was provided in terms of the nine deflections measured by the nine geophones as part of FWD equipment. These are distanced at varying positions from the centre of the load, typically at 0mm, 200mm, 300mm, 450mm, 600mm, 900mm, 1,200mm and 1,500mm. As part of this research D0, D300, D600 and D900 were needed. Table 4.4 summarises the required FWD parameters and their formulae. The formulae were used to determine their values and stored in a separate spreadsheet.

Table 4.4 FWD parameter formulae

FWD parameter	Formula
D0	D0 as measured at the point of loading
RoC	$ROC = \frac{200^2}{((2D_0)(1-(D_{200}/D_0))}$
BLI	BLI = D0 – D300
LLI	LLI = D600 – D900

For all three RCAs, condition data for rutting and roughness was of interest. An exception to table 4.3 is rut rate which was not supplied but calculated. The condition data was recorded in RAMM in each wheelpath for both the left and right lanes (labelled as L1 and R1 with left lane being the increasing direction) for treatment lengths every 20m. In comparison for each treatment length recording FWD data, data was measured for either the left or right lane (left lane being the increasing direction). The distance at which FWD data was measured along each carriageway differed for each RCA and these are listed below in table 4.5. As shown, the distances for which TRC FWD data points were measured varied between 150m and 200m along the road carriageway.

Table 4.5 Distances used for FWD and SI data collection

RCA	FWD distances (m)	SI distances (m)
MCC	20	20
TRC	30	≈150–200
NSCC	80	80

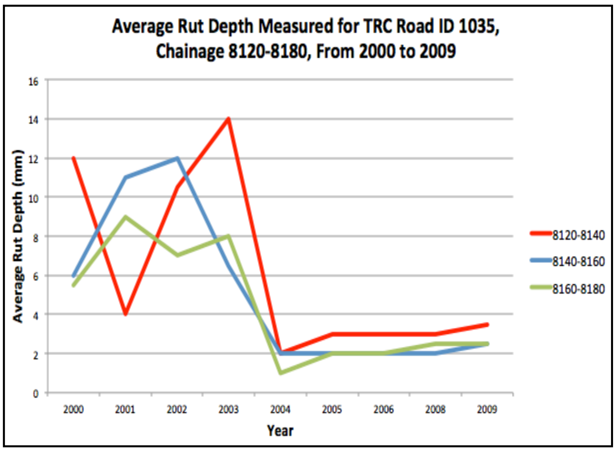
Treatment lengths were recorded for different years and listed according to the road ID number. As a result the analysis period was from 2003 to 2011 for the MCC data, from 2000 to 2009 for TRC, and from 1999 to 2011 for the NSCC data. The data was sorted according by the road identification number assigned to each road name in RAMM (road ID). As an indication, across all three RCAs approximately 183,000 data points for rutting were processed alone.

4.2.2 Data processing

Historical rutting condition data for each RCA was sorted for each road ID in increasing year order. A diagnostics process was then undertaken to determine which pavement sections had undergone rehabilitation. This was visually assisted by line graphs, which were drawn for each treatment length detailing the average rut depth (mm) against the year of measurement. The procedure was then carried out as follows:

- Graphs were visually scanned for sudden variations. An example is highlighted in figure 4.1 where 2003 is the year in which the pavement was rehabilitated hence causing the lower values observed in 2004 (note that similar low values for chainage 8120 to 8140 measured in 2001 were considered as outliers caused by measurement error).
- The graph was compared with other treatment lengths of the section – treatment lengths preceding and succeeding the treatment length in question. This assisted in deducing a pattern to explain pavement behaviour of the road section and ensure there were no outliers.

Figure 4.1 Example of a line graph generated for determination of rehabilitated pavement sections



Abnormalities in the data were common, for example in the NSCC data measured in 2009 there was a sudden peak observed for different road IDs. This indicated a possible measurement or equipment error in 2009.

The rut rate was calculated for rehabilitated pavement sections with a rut history. Using the line graphs generated, an average rut rate as expressed in millimetres per year was calculated as the slope of the line over the years leading up to rehabilitation.

The condition data for the identified rehabilitated road sections were collated and stored on a separate spreadsheet. The SIs, FWD parameters and rut rates were then inserted into the condition data spreadsheet and matched with the condition data according to the location (chainage) of each road ID and lane.

4.2.3 Data analysis

Analysis of the data was undertaken in two stages:

1. Calculation of correlations between pavement condition data, SIs and their corresponding FWD parameters
2. Testing predefined boundary condition values and a deduction of the pavement condition for the three RCAs.

The first stage was undertaken to determine the parameters defining critical failure, boundary values and to introduce strength parameters as decision criteria for the TSA. It was to compare base or unprocessed FWD data with strength indices and involved plotting SIs against their corresponding condition and FWD parameters as scatterplots. Parameters with correlations, as detailed in section 4.2.4, were then selected for use in the improved TSA.

The second stage was undertaken to test predefined boundary condition values for FWD parameters and to define the pavement condition for each RCA. It involved plotting each group of significant FWD parameters into histograms. The distributions were then grouped into three categories according to their structural condition – sound, warning and severe. These categories were defined according to similar research conducted by Horak (2008, p4) and are reproduced in this report as table 4.1.

4.2.3.1 Data availability

Of all the data recorded for the three RCAs, Manukau was deemed the best for deducing correlations due to the availability of all three data types required. Hence all correlations between SIs and FWD parameters were deduced from the MCC data. It is noted that no correlations were deduced between condition data and SIs and so are not detailed in this section. TRC had an excellent wealth of data for the condition and FWD parameters; however, SIs were only measured in 2010 and hence there were insufficient parameters to deduce correlations. NSCC data had a moderate sample size of rehabilitated sections for analysis; however, there was insufficient FWD data to deduce any relationships.

Due to insufficient amount of data available to analyse for correlations, no significant correlations were found related to BLI and LLI.

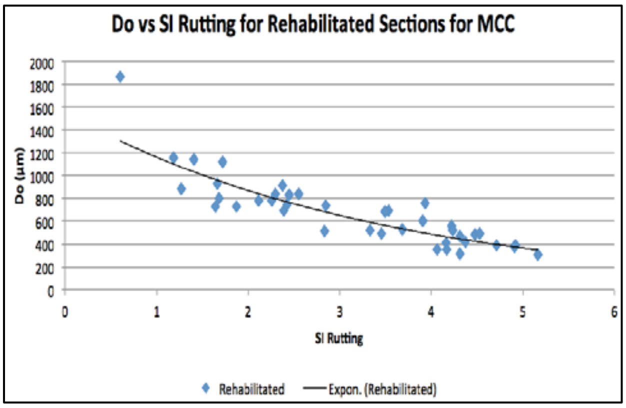
4.2.4 Correlations between FWD parameters and SIs

Parameters showing strong correlations indicated that the base FWD data alone would suffice as a decision criterion in the TSA. (While it would have been preferable to use the SIs directly, these are not yet in the public domain for use on any network with FWD data. There is also some analysis required to determine them. Therefore the decision was made to utilise direct FWD data into parameters rather than calculate indices which were not readily available. Should this situation change, the parameters can be easily substituted.) Of all the parameters analysed it was shown that the use of D0, SIrutting, SIflexure and RoC would be of most interest as there were clear correlations. This section details the correlations observed and hence is of interest for the improved TSA.

4.2.4.1 D0 versus SIrutting

The strongest correlation seen was between D0 and SIrutting which is shown in figure 4.2. A decreasing relationship was found between the two parameters. In general as SIrutting increased the deflection D0 decreased. D0 is the maximum deflection of the pavement as measured directly beneath the loading plate of an FWD apparatus and gives a good indication of the overall pavement strength. SIrutting gives an indication of the pavement's structural resistance to rutting and is associated with subgrade failure. The correlation observed between the two parameters was as expected because a higher D0 is associated with higher rut depths, which in turn is associated with lower SIrutting values. This infers that, in general, pavements with a greater structural resistance to rutting have lower deflections. The observed correlation demonstrates that for implementation into the improved TSA, D0 alone will suffice for use as a decision parameter for pavement rehabilitation regarding subgrade failure. When a problem in rutting is encountered, one needs only to refer to D0 as a decision parameter for pavement rehabilitation for subgrade failure.

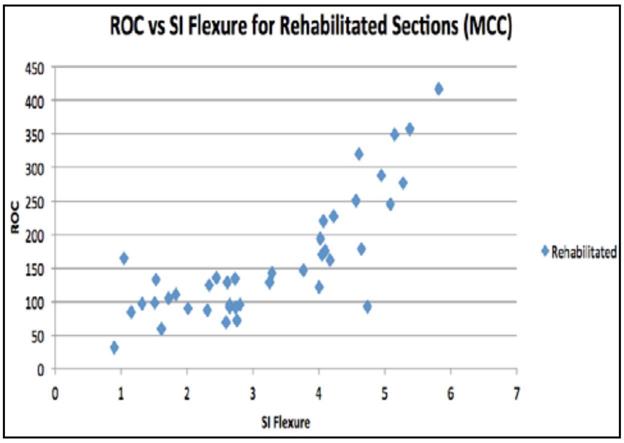
Figure 4.2 Plot of D0 versus SI rutting



4.2.4.2 RoC versus SIs

Another clear correlation observed between parameters was between RoC and SIflexure as illustrated in figure 4.3. An increasing relationship was found between the two parameters. RoC expresses how curved the deflection bowl is (as measured by a FWD) and depends upon which zone it is taken from. The deflection bowl passes through three distinct zones, zone one being closest to the point of loading and zone three being furthest away (Horak 2008). Zone one is associated with the upper pavement layers as opposed to the subgrade and this is further inferred through the formula used to calculate RoC values in this research. In general a higher SIflexure is associated with higher RoC values. This infers that weak upper pavement layers are manifested by low RoC values within zone one. SIflexure, or fatigue such as cracking, gives an indication of the resistance of the upper pavement layers to flexural distress. Therefore lower SIflexure values indicate a low resistance to flexural distress and the corresponding lower RoC values are a result of higher D0 values.

Figure 4.3 Plot of RoC versus SIflexure

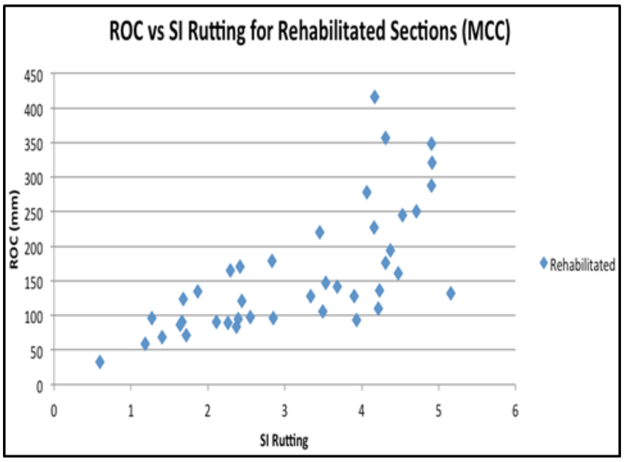


Through observation two correlations were perceived. At a SIflexure value from approximately 4 upwards, values for RoC increased at a much higher rate than for values less than 4.

For use of decision parameters for pavement rehabilitation for upper pavement layers in the improved TSA, RoC alone should suffice; however, this should be tested further with abundant RCA data.

A similar but less cohesive correlation was observed between RoC and SIrutting as shown in figure 4.4.

Figure 4.4 Plot of RoC versus SIrutting



A general increasing relationship was found between the two parameters indicating the higher the SIrutting, the lower the RoC. As mentioned in section 4.2.4.1, SIrutting is associated with subgrade failure. Hence the correlation observed above is not as consistent as the correlation found between RoC and SI flexure.

As with the relation shown in figure 4.4 a similar correlation was deduced, with values for RoC accelerating from a SIrutting value of approximately 4. However, due to the scarcity in data points the relationship could not be confirmed. Therefore for implementation into the improved TSA, both SIrutting and RoC should be looked at together for use as decision parameters for pavement rehabilitation for subgrade failure.

4.2.4.3 RCA pavement condition

Of the significant FWD parameters highlighted in section 4.2, distributions for both MCC and TRC were mapped out and zoned according to the criteria defined in table 4.1. No adjustments were needed for the boundary values and were accepted for New Zealand conditions. For the NSCC FWD data was insufficient and hence no distributions could be drawn.

Figure 4.5 shows a comparative distribution of pre-treatment D0 between MCC and TRC respectively. For the majority of each distribution approximately 90% of rehabilitated TRC sections had severe D0 values, whereas about half of the MCC sections had D0 values in the warning zone.

The Taranaki region is known to have pumice issues in the subgrade therefore making the pavements more flexible. Therefore it is possible that the high number of D0 values within the severe zone were a result of this, as TRC pavements are known to be long lasting for their strength built for the high volumes of traffic carried.

Figure 4.6 shows a comparative distribution of RoC between both RCAs. Approximately 90% of the MCC rehabilitated sections had sound RoC values and 70% of TRC sections had values in the warning zone. From these values alone it indicated that rehabilitation of MCC pavements were not due to upper pavement problems. However in figure 4.5 for the MCC data there was no obvious sway of D0 data sitting within a certain condition zone. The only explanation for this was a possible variability in subgrade. However this highlights a concern and raises questions on the deciding factors used to determine rehabilitation of pavements within the region. It further highlights the importance of the improved TSA to guide these decision criteria.

Finally, no inference could be made regarding pavement behaviour for the NSCC data. The lack of FWD data for analysis showed that decisions for the RCA were being based upon visual surveys. This highlights a trend in practice used nationally among many RCAs.

Figure 4.5 Comparative distribution of D_0 for MCC and TRC

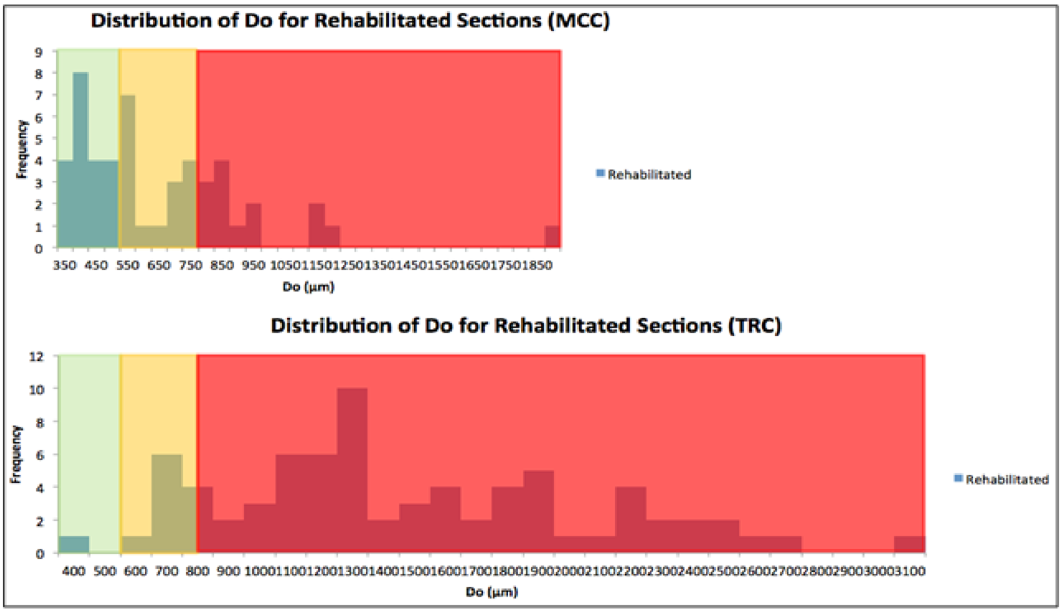
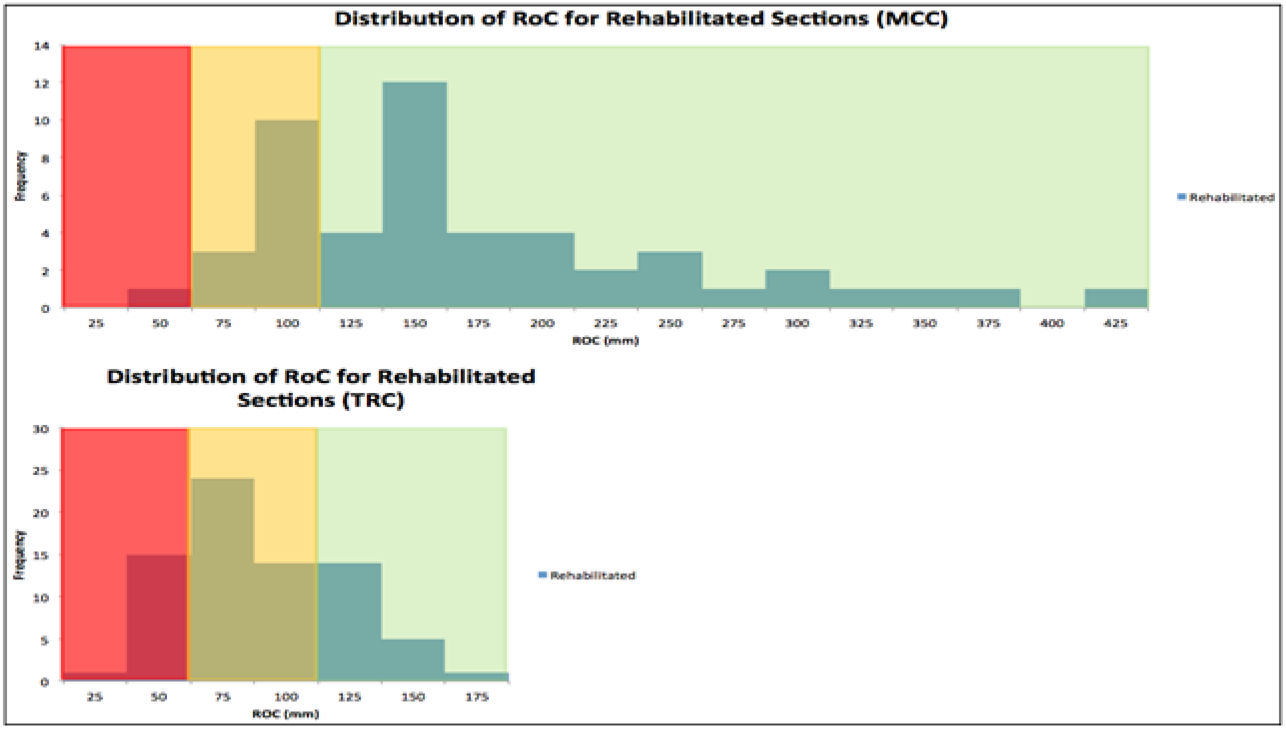


Figure 4.6 Comparative distribution of RoC for MCC and TRC



4.2.4.4 Rut rate

Rut rate was plotted against SIrutting and D0 using MCC and TRC data respectively, according to the data available. There was a spread of rut rates across the spectrum but the important observation made was that many sites with rut rates less than 1mm/year were being rehabilitated. However the sites with the highest rut rates belonged to those pavements on the weaker end of the spectrum.

Figure 4.7 Plot of D0 versus rut rate for TRC

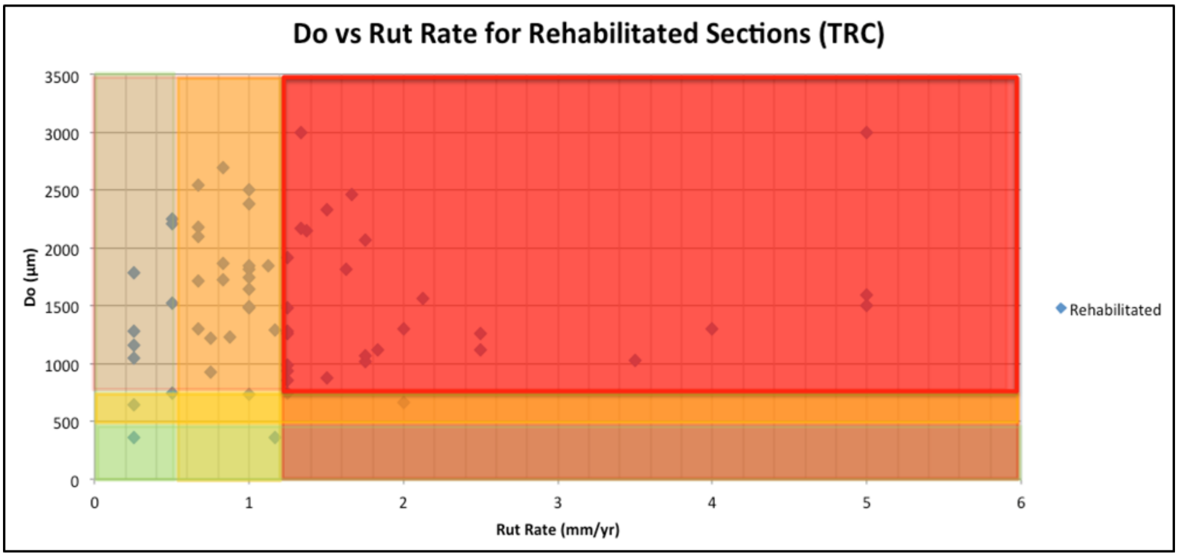


Figure 4.7 shows the plot of D0 versus rut rate for TRC. Using the condition ratings as defined in table 4.3 for D0 and an approximation according to the distribution of data for rut rate, it can be seen that severe sections in TRC have a rut rate lower boundary value of 1.2mm/year. Overall these results showed that rutting was not a driver in the rehabilitations made as rehabilitation sites were triggered independent of rut rate. This is in contrast to the FWD information where very few sites had D0 values less than 1mm.

4.2.4.5 Further required work

This research has shown that sufficient data is required to achieve more robust and reliable inferences regarding the possible parameters for use in the improved TSA and the values defining them as decision criteria for pavement rehabilitation needs. Therefore **it is recommended** that sample sizes from other RCAs with a sufficient number of data points for all three data types be analysed. Through analysis, confirmation of the correlations obtained in this research will be acquired, as well as the possible discovery of other useful correlations for use in the improved TSA. In addition, through the strength of the correlations, criteria for the implemented parameters defined as the critical indicators for rehabilitation treatments should be obtainable.

In addition, parameters deemed significant and their critical values indicating the need for rehabilitation as referred to in section 4.2.4 should then be tested on a New Zealand road network to confirm their robustness and reliability for implementation into the improved TSA.

4.3 Use of HSD data for treatment option selection

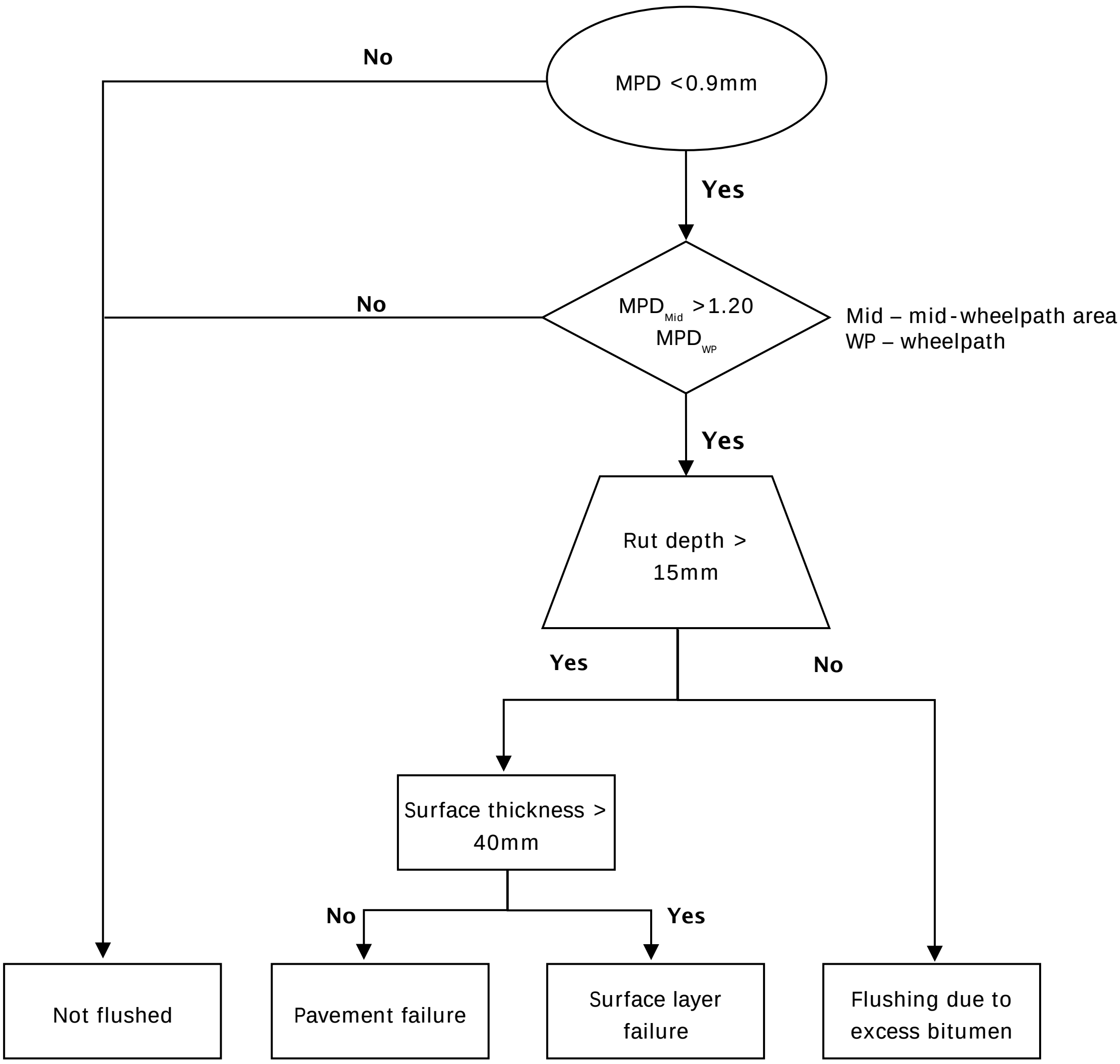
The analysis presented in the above sections was carried out to obtain more accurate identification of treatment selection on chipseal surfaced pavements using FWD data. However, for sections where no FWD data is available, HSD can be utilised to determine a failure mechanism and therefore pavement renewal treatment option.

The following decision tree, originally developed as a failure mechanism for flushing (Kodippily 2011), has been adopted as a mechanism for pavement options utilising the texture data from HSD. The aims of this study were to utilise area-wide pavement treatment data, HSD and pavement performance data from the long-term pavement performance programme to develop a mechanism to easily identify pavement sections that are flushed or at risk of flushing, and determine the best maintenance strategy for the sites.

Based on the study findings, a diagnostic approach was developed to aid in determining the most likely causes of flushing on pavements. The results give a trigger mechanism for identifying treatment lengths where flushing signifies a rehabilitation treatment is required. It will also serve to cover seal layer instability as a failure mechanism where rutting is present and the surfacing is greater than 40mm thick. This work on determining the failure mechanism for seal layer instability will require more work before a more definitive test may be included into the TSA mechanism.

The decision-making process resulting from the data analysis is shown in figure 4.8.

Figure 4.8 Decision tree for selecting treatment options



4.4 Where no FWD data or HSD available

It is common for many local authority networks to use only visual rating data and roughness data as measures of condition. Often there is little or no FWD data or HSD available. The PV calculation, however, requires an analysis to determine the type and therefore cost of the pavement renewal treatment. It has been used where no FWD or HSD data is available from which to infer whether a basecourse treatment or full pavement renewal is required.

The selection of treatment option is dependent on the location of the site:

- urban locations
  - <10,000 vpd and/or collector or below = assume basecourse improvement
  - >10,000 vpd and/or arterial or above = assume pavement improvement
- rural locations
  - assume basecourse improvement.

5 Improvements

A number of improvement areas were identified in the process to bring it up to date with current practice, resolving some of the user issues and utilising advances in technology and data availability since the last improvements over 15 years ago.

The overall process will remain similar but at each of the various steps, changes and improvements are discussed and recommendations made. This section focuses on each step individually and considers the changes and recommended improvements.

5.1 Step 1: Compute area treatment costs

Before undertaking any of the four possible treatment types listed below, the costs of necessary preliminary maintenance and drainage repairs are calculated.

The four treatments are

- general maintenance
- resurface
- smoothing (smoothing overlay)
- strengthening (rehabilitation).

The preliminary repair costs are estimated from deficiencies revealed in the pavement condition rating.

5.1.1 Calculation of general maintenance costs

The current TSA process calculates general maintenance costs based on the latest condition rating data unless the section has been resurfaced since the condition survey. However, previous maintenance activity can mask deterioration by removing faults through patching and remedial works. Thus a section may have been in a poor condition and resurfacing should have been undertaken earlier but, through maintenance activity, there are few or no faults found (see step 3).

It is recommended that historic maintenance costs be included when testing the current condition to check whether the life of the current surfacing has been extended beyond its optimum intervention**.** The previous year's maintenance costs plus recorded patches should be used to undertake this test. The use of pavement and surfacing activity classes only is recommended.

It is recommended a cost set item for stabilised patches is added to the cost set structure.

Currently, there is no consideration of thin asphaltic surfacings on flexible granular pavements. **It is recommended** that a new cost set table be added for thin asphaltic surfacings as their routine maintenance costs are different from those for chipseal surfacings. The thin surfaced flexible cost set table should be split into two parts, one for chipseal and one for asphaltic surfacings. The asphaltic surfacing cost set would need to be developed on a basis of repairs for relevant faults.

The TSA calculates different general maintenance costs prior to each treatment type. For example, prior to a reseal, the costs of repairing all faults are included while for a strengthening treatment, no repair costs are included prior to treatment as the pavement is to be renewed. The preliminary general maintenance costs for asphaltic surfacing will include only the repairs for shoving, as per the current smoothing overlay.

5.1.2 Calculation of area treatment costs

5.1.2.1 Resurfacing treatments

For resurfacings, equation 5.1 should be retained:

*Costs = treatment length * carriageway width * unit cost (reseal)* (Equation 5.1)

Currently. however, there is no rate for thin asphaltic surfacings and this should be added on a similar basis to the chipseal treatment cost.

In addition, consideration has been given to a user defined trigger as to when the asphalt surfacing will apply. This could include similar options to the current NZdTIMS options as follows:

- 'like for like', ie maintain asphalt surfacing if currently surfaced in asphalt
- user-defined traffic trigger, eg >15,000 vehicles per day will trigger an asphaltic resurfacing over a chipseal treatment.
- combination of both the above, eg asphalt resurfacing shall be applied where already present or traffic exceeds 15,000 vehicles per day. .

While this has been considered, **it is recommended** that a simple like-for- like policy be maintained within the TSA. First, the purpose of the TSA is as a candidate selection tool and by setting triggers to predict asphalt or chipseal, we enter the realm of design which is not the purpose of the TSA. Second, the user-defined trigger creates issues with comparing across networks. There are also still 'engineering factors' that come into play which are difficult to account for such as high stress or amenity areas, eg cul de sac heads, intersections or industrial areas.

5.1.2.2 Pavement treatments (urban)

For SCT treatments, there are currently two options: strengthening and smoothing. **It is recommended** that these be replaced with a modified basecourse treatment or a full pavement renewal. The modified basecourse treatment is where strengthening is required to address substandard basecourse performance. The full pavement renewal is required where the pavement is of insufficient depth and quality to protect the subgrade.

For urban treatments, it is assumed that kerb and channel constraints overlay treatments and therefore inlay treatments only are possible.

For a basecourse modification, the cost equation is:

*Costs = treatment length * carriageway width * unit cost per m^2 (basecourse improvement)* (Equation 5.2)

There are two cost scenarios the unit rate must consider and the user provides:

- modified basecourse treatment with chipseal surfacing
- modified basecourse treatment with asphaltic surfacing.

Typical urban basecourse modification treatments would include rip and remake or stabilisation treatments.

For a full pavement renewal, the cost equation is:

*Costs = carriageway length * carriageway width * unit cost per m² (full pavement improvement)* (Equation 5.3)

There are three cost scenarios the unit rate must consider and the user provides:

- pavement improvement treatment with chipseal surfacing
- pavement improvement treatment with asphaltic surfacing
- pavement improvement treatment utilising structural asphalt construction.

The second option operates off the same asphalt surfacing trigger criteria described above, ie the 'like for like' basis.

The shift to a full structural asphalt construction would apply to a traffic trigger, for example greater than 20,000 vpd and/or quantity of heavy vehicles. This would be user defined to allow networks to better align expenditure forecasts from the TSA to decisions that might have to be made in the field.

Typical urban pavement renewal treatments would include mill and replace treatments which may comprise stabilisation of sub-base or subgrade layers and utilise granular or structural asphaltic pavement layers. More comprehensive stabilisation treatments maybe utilised such as foamed bitumen. The objective is to strengthen the pavement as a whole. The triggers and cost rates will depend on what each RCA utilises to suit the characteristics of its network.

5.1.2.3 Pavement treatments (rural)

As above there are currently the two options, strengthening and smoothing, and **it is recommended** these be replaced with a modified basecourse treatment or a full pavement renewal. This reflects the issue of pavement renewals required due to either basecourse failure or a subgrade failure. This is discussed in more detail in chapter 4. For rural treatments, it is assumed there are no constraints on overlay treatments. In addition, shoulder treatments need to be included in the cost calculation.

The rural basecourse modification is based on the current smoothing overlay treatment assumption of a nominal 70mm overlay on high spots that average 100mm in depth. The width of metal includes the carriageway width, LHS and RHS shoulder widths plus an allowance of 1.5m each side to allow for placing metal down the feather edge. The cost equation is:

*Overlay cost = carriageway length * (carriageway width plus shoulder width (LHS & RHS) + 3.0m) * unit cost per m³ (granular basecourse in place) * 0.1* (Equation 5.4)

*Surfacing cost = carriageway length * carriageway width * unit cost per m² (first coat)*

There are two cost scenarios the first coat unit rate must consider and the user provides:

- first coat chipseal surfacing
- first coat asphaltic surfacing (which may include a membrane seal).

The second option will operate off the same asphalt surfacing trigger criteria described above, ie either a 'like for like' basis and/or some level of traffic volume trigger.

The full rural pavement renewal is based on the current strengthening overlay treatment assumption of a nominal 150mm overlay. The width of metal includes the carriageway width, LHS and RHS shoulder widths plus an allowance of 1.75m each side to allow for placing metal down the feather edge. The cost equation is:

*Overlay cost = carriageway length * (carriageway width plus shoulder width (LHS & RHS) + 3.5m) * unit cost per m^3 (granular basecourse in place) * 0.15* (Equation 5.5)

*Surfacing cost = carriageway length * carriageway width * unit cost per m^2 (first coat)*

There are two cost scenarios the first coat unit rate must consider and the user provides:

- first coat chipseal surfacing
- first coat asphaltic surfacing (which may include a membrane seal).

The second option will operate off the same asphalt surfacing trigger criteria described above, ie either a 'like for like' basis and/or some level of traffic volume trigger.

The structural asphaltic option is not often viable in a rural location and, as with the current configuration, is not included as an option.

5.1.3 Calculation of drainage treatment costs

No changes are recommended for the calculation of drainage maintenance costs utilised in the current set up.

5.2 Step 2: Assess the need for resurfacing

5.2.1 Current TSA assessment criteria

The criteria for determining the need for a resurfacing treatment is as follows (NZIHT 1999):

Second coat seal:

- surface function = coat1 (first coat) and pavement use >2
- surface function = coat1 (first coat) and pavement use >= 2 and surface age >1 year

Reseal in budget:

- percentage wheelpath shoved >3%
- percentage wheelpath cracked >3%
- percentage area scabbed > 25% and top surface age >50% *top surface expected life
- no. of potholes (includes pothole patches) per lane km >50
- combined percentage wheelpath shoved and cracked >3%
- percentage wheelpath flushed >30%
- sideway-force coefficient routine investigation machine (SCRIM) deficiency for entire treatment length >= SCRIM deficiency threshold

- SCRIM deficiency for site category >= SCRIM deficiency threshold
- SCRIM deficiency for continuous failed length >= SCRIM deficiency threshold.

Reseal next time:

- percentage wheelpath shoved >1%
- percentage wheelpath cracked >1%
- percentage area scabbed >10% and top surface age >50% * top surface expected life
- no. of potholes (includes pothole patches) per lane km >25
- combined percentage wheelpath shoved and cracked >1%
- percentage wheelpath flushed >15%
- surface life expectancy factor >user defined limit[1].

Locking coat seal:

- percentage area scabbed >10% and top surface age <50% * top surface expected life.

It should be noted the SCRIM tests for reseal in budget are optional and can be switched off by the user.

It should also be noted that not all faults may lead to a reseal fix in the field. However, the purpose of the TSA is to identify candidates for inspection, albeit with the expectation of providing a meaningful assessment of renewal need in the next two years.

This process also assumes treatment length boundaries are appropriate.

5.2.2 Accounting for maintenance costs

The current selection criteria do not account for the situation whereby maintenance costs have masked the condition of the treatment length.

RAMM stores maintenance cost information in the maintenance cost table broken down into activity areas. The two most relative to this analysis are pavement and surfacing.

Consideration of the use of the maintenance cost data raises several questions:

- What activity areas to cover? Pavement and surfacing activities are the simplest to use. Other possible options are drainage and shoulder activities or repair categories within these activities are a possibility. **It is recommended** only pavement and surfacing are used as these are directly relevant to the outcome from the TSA.
- What level of granularity to consider?

[1] The surface life expectancy factor allows for where the surface condition does not trigger any treatment criteria but the surface has exceeded its expected life. The user can specify the percentage by which the surface age may exceed the expected surface life before the reseal next time flag is set. This mechanism was added to the TSA process after the 1997 improvements.

  - One option is simply to combine pavement and surfacing costs into a single trigger for reseal in budget and reseal next time.
  - A consideration is to further identify maintenance cost triggers on pavement costs as to whether a pavement renewal is required. If maintenance costs are predominantly pavement faults then a pavement issue may be prevalent. If maintenance costs are predominantly surfacing only, this may indicate the pavement capacity is adequate. A further factor is surface age. If either faults are present when the surfacing is still young, this may suggest pavement strength is poor.
- How far back in time to be considered? **It is recommended** one year of maintenance costs be considered. The test is to check where maintenance has masked the underlying condition. In a case of no maintenance being done due to a rehabilitation treatment programmed, then condition should still show faults as being present. If a renewal treatment is programmed, this should still be on the list for field validation if for some reason TSA does not recommend a treatment.

For this part of the analysis, the trigger is simply to determine whether a renewal treatment is required in the next two years. There is no separation at this stage between pavement and surfacing renewal. The requirement for a pavement renewal comes in step 2a. It assumes that maintenance cost data is available. If this is not so, the test is not applied. However, with the increasing use of RAMM Contractor, maintenance cost data is becoming more comprehensive and more commonly used.

It is recommended the following test be added:

Reseal in budget:

- Pavement and surfacing maintenance costs for the 12 months prior to three months from the analysis date >6% of surface area * stabilised patch rate AND surface age >3 years.

Reseal next time:

- Pavement and surfacing maintenance costs for the 12 months prior to three months from the analysis date >2% of surface area * stabilised patch rate AND surface age >3 years.

The slightly higher percentages are to allow for some conservatism in the use of the maintenance cost test. It also allows for the fact that some repairs will be digout patches which are more expensive. The three month period allows for maintenance costs to be entered into RAMM subsequent to the activity being carried out on site. The recommendations do require validation for confirming the percentage quantities but the concept is recommended to be implemented.

5.2.3 Accounting for thin asphaltic surfacings

The current selection criteria do not account for thin asphaltic concrete surfacings on thin surfaced flexible pavements**. It is recommended** that the following test be added:

Reseal in budget for thin asphaltic concrete surfacings:

- percentage wheelpath shoved >10%
- percentage wheelpath cracked >10%
- percentage wheelpath joint plus longitudinal and transverse cracking >40%
- percentage area scabbed >40%

- combined percentage wheelpath rutted >20%
- percentage wheelpath flushed >20%.

Reseal next time for thin asphaltic concrete surfacings:

- percentage wheelpath shoved >5%
- percentage wheelpath cracked >5%
- percentage wheelpath joint plus longitudinal and transverse cracking >20%
- percentage area scabbed >20%
- combined percentage wheelpath rutted >10%
- percentage wheelpath flushed >10%
- surface life expectancy factor >user defined limit.

The above factors are based on the structural asphaltic concrete condition criteria for a resurfacing in budget or resurfacing next time. The intervention criteria are significantly higher than that for a chipseal surfacing which follows common practice. Testing will be needed to validate any values to be introduced into TSA.

5.2.4 Introduction of a composite index

Currently the TSA utilises a series of discrete parameters to select the treatment options, as detailed in section 5.2.1 for reseal in budget. There is no use of composite indices where a small amount of distress across a number of distress types could trigger a treatment. The concern is that under the current system such a site is not flagged as requiring treatment as none of the individual parameters have progressed sufficiently to trigger a treatment.

There is scope for further investigation on whether a composite index would provide any additional benefit. The first aspect is quantifying what proportion of treatment lengths would fall into this category and whether the inclusion of the composite index would create a benefit to justify the alteration. COST (2008) details the calculation of composite indices for road pavements and advises on their development. The second aspect is quantifying how many treatment lengths would fall into this category and whether the inclusion of the composite index would create a benefit to justify the alteration.

5.2.5 Adding a year 3 selection criteria

To create a three year FWP from the TSA process, in line with typical current practice across RCAs, the simplest implementation would be to amend the surface life expectancy factor test from the ‘reseal next time’ flag and use it to populate the’ reseal next time plus 1 year’ category. This has the advantage of retaining the current trigger assessment criteria for consistency with historic analyses.

Given the additional year that treatments are considered for application, the trigger should be able to consider surfacings not yet past their expected life. This may require the shift from the current ‘extra over’ measure, ie surfacing exceeds expected life by 20% to a whole-of-life measure whereby the surface age is 120% of the expected life. This would allow a treatment at say 90% of the surfacing’s expected life but in good condition to be triggered in the third year.

An alternative could be to allow a negative number. For the example given above at 90% of its expected range could be allowed to accept -10% exceedance of expected life.

However, without any forecasting of condition, such a three year criteria is difficult to achieve. While a practitioner may be able to make a visual assessment in the field, a desktop assessment will typically only be made around how close the surfacing is getting to its expected life. **It is recommended** that the two year assessment window remains unchanged.

5.2.6 Accounting for road classification

The current TSA process has a single trigger level independent of road class. The only indirect parameter currently indicating road classification would be the use of expected surface lives which vary by pavement use. This is indirect in that it assumes a high classification road would have a shorter lifecycle than a lower classification road. However this trigger only applies to locking coats or the surface life expectancy factor.

The New Zealand roading industry is moving towards a national road classification system through the 'one network road classification'. In line with this, the Road Efficiency Group will need to look at varying levels of service and customer and technical measures across these classifications. In a similar vein, the Transport Agency, through the network outcome contracts, wishes to similarly vary intervention criteria and levels of service across the state highway classification categories.

In the NZdTIMS setup, intervention criteria such as cracking, rutting and roughness typically vary across functional pavement groups.

Given these changes in the maintenance environment, it would seem beneficial for the TSA process to allow for customisation according to road classification, or at the very least traffic volume.

A simple method would be to have a user-defined table, similar to the unit cost sets which are populated with standard default settings for each road classification. These values could either be user defined or hard coded as are the current TSA triggers. As stated before, the use of user defined triggers does make it difficult to compare results between networks.

An unsubstantiated potential example is given below for state highways and local roads.

Table 5.1 Reseal in budget assessment for chipseal surfacings on state highway classification

Parameter	National strategic	Regional strategic	Regional distributor	Regional collector	Reseal next time test*
Shoving	3%	3%	4%	5%	50% of reseal in budget triggers
Alligator cracking	3%	3%	4%	5%	50% of reseal in budget triggers
Shoving plus cracking	3%	3%	4%	5%	50% of reseal in budget triggers
Potholes/km	25	35	40	50	50% of reseal in budget triggers
Flushing	10%	10%	15%	20%	50% of reseal in budget triggers
Scabbing	20	25	30	35	50% of reseal in budget triggers

*The reseal next time test is set at 50% of the reseal in budget triggers. For example reseal next time trigger for regional distributor classified section would be cracking exceeding 2% as it is 50% of 4%, the reseal in budget trigger.

Table 5.2 Reseal in budget assessment for chipseal surfacings on local road classification

Parameter	Strategic	Arterial	Collector	Local	Reseal next time test*
Shoving	3%	4%	5%	6%	50% of reseal in budget triggers
Alligator cracking	3%	4%	5%	6%	50% of reseal in budget triggers
Shoving plus cracking	3%	4%	5%	6%	50% of reseal in budget triggers
Potholes/km	35	40	50	60	50% of reseal in budget triggers
Flushing	10%	15%	20%	30%	50% of reseal in budget triggers
Scabbing	25	30	35	40	50% of reseal in budget triggers

*The reseal next time test is set at 50% of the reseal in budget triggers. For example reseal next time trigger for an arterial classified section would be cracking exceeding 2% as it is 50% of 4%, the reseal in budget trigger.

5.3 New step 2a: What SCT treatment is required?

5.3.1 SCT selection

It is important to note that this is a simplified analysis based on very limited information. Traffic loading and pavement material characteristics for example are not involved. The process is a simple desk-based analysis using information available to identify a possible failure mechanism and therefore proposed treatment for identifying candidate sites. It will not replace a more detailed engineering-based assessment of pavement performance to determine whether a pavement renewal is viable and what option is best.

First, it is to be assumed that a pavement renewal treatment will only be triggered if the treatment length first meets the criteria for a resurfacing treatment, as described in the process in section 5.2. This is an important change from the previous set up. Rehabilitation treatments were assessed based on the economic justification via a benefit-cost calculation. While a treatment length may have justified an SCT under this assessment, it does not necessarily mean that is the optimum time to intervene. Therefore, **it is recommended** the treatment length condition must be able to trigger a surfacing renewal to trigger a pavement renewal. Otherwise the shape correction pavement renewal treatment is deferred.

This change is partly driven by a change in the Transport Agency's economic justification policy. At the time of the previous set up, the justification was driven by a benefit-cost calculation. VOCs have now fallen away from the justification process with the assessment being a simpler PV-based calculation. Therefore, timing should be assumed to align with the need for a surfacing intervention as a minimum to providing the optimum intervention timing.

Second, the type of option required needs to be determined so the capital cost of the treatment can be estimated as accurately as possible. Under the previous set up, two options were available:

- Smoothing (smoothing overlay): Assumed that only reduction in roughness will be achieved and no strengthening takes place. Future resurfacing lifecycles and maintenance cost streams are unaffected by intervention.
- Strengthening (rehabilitation): Assumed that strengthening of the current pavement structure takes place with reduced (reset) roughness values, extended future resurfacing lifecycles and lower maintenance cost streams.

The smoothing treatment is now obsolete as VOC are no longer a benefit in the economic assessment of treatment options. Therefore the only intervention is a pavement strengthening. The cost options for the pavement strengthening are detailed in section 5.1.2.

For pavements requiring a treatment, the type of failure mode is important for allowing the treatment option to be qualified.

It is recommended that treatment options are determined based on two mechanisms:

- failure in the upper layers, typically due to shear failure in the basecourse
- failure due to insufficient protection of the subgrade, typically as a result of excessive subgrade strain

Where FWD data is available, a combination of RoC and central deflection will be used to determine the failure mode and therefore treatment option for pavement renewal. This is detailed in section 4.2, tables 4.1 and 4.2. This work may be developed further through the work in progress by Graham Salt looking at a summary of historic FWD results across the country.

Where HSD is available, **it is recommended** that the flushing test outlined in section 4.3, figure 4.8 is utilised.

Where no such data is available, the following test is applied:

- urban locations
  - <10,000 vpd and/or collector or below = basecourse improvement
  - >10,000 vpd and/or arterial or above = pavement improvement
- rural locations
  - basecourse improvement

5.4 Step 3: Estimate resurface cycle times

5.4.1 Current process

Under the current TSA, when dealing with a very high existing surface distress, a check is made to see if some of the distress could have been economically averted by shortening the resurfacing cycle by one or two years. If so the future resurfacing cycle is adjusted accordingly.

The maintenance repair cost is calculated as the sum of all pothole and shoving repairs (NZIHT 1999, p45). **It is recommended** this be amended to include alligator cracking repair costs also.

The life of the future surfacing is generally estimated from the performance of the current surfacing. In the case of premature failure of the current surfacing (less than 70% of the expected life for the traffic level), the condition of the pavement is checked. If the drainage condition indicates drainage is seriously deficient, the assumption is made that drainage improvements will partially restore the expected surface life achievement in the future.

The types of surface deficiencies are also checked and if these are not pavement related then the pavement is assumed to be capable of supporting future surfacing for a normal lifespan.

If the drainage is not deficient and the distress types do not indicate a design or construction fault, the pavement is assumed to be incapable of supporting future surfacing for normal life spans. Consequently the pavement will have a short resurfacing cycle and high maintenance costs, which will significantly increase the PV of future maintenance and resurfacing.

5.4.2 Accounting for maintenance works

Similar to section 5.2.2, the current TSA selection criteria does not take into account a situation where maintenance costs have masked the condition of the treatment length by treating the faults that were present. As the TSA uses the condition data in RAMM, maintenance activity prior to the condition assessment will repair the faults. The purpose of checking the condition data is to ascertain if the surfacing has deteriorated early. If maintenance has taken place on the treatment length, the condition can appear to be good and a treatment is not triggered to repair the underlying distresses. It is therefore important to check the level of maintenance expenditure as well.

It is recommended that the following test be added:

- Pavement and surfacing total maintenance costs for the 12 months prior to the 3 months before the analysis date > cost of reseal due to seal delay.

5.5 Step 4: Compute the present value of future maintenance

5.5.1 Present TSA process

The TSA currently computes the discounted PV of future reseals and general maintenance activities for each of the four area options over a 25-year period.

The general maintenance costs are not estimated on any theoretical basis but are assumed to occur mainly in the years immediately before each resurfacing, building up to a peak in each resurfacing year. Hence the general maintenance PV is a function of the length of resurfacing cycle only.

The discount rate used is 10% and is not alterable by the user.

5.5.2 Review of the current calculation

NZIHT (1999) outlines the methodology behind the calculation of PV future maintenance sums. It is not the intention to repeat that process in this report. However it is important to note that the calculation was revisited to determine the effect the change in discount factor from 10% to 6% would have on the results. This revisit was not straightforward due to errors in several of the equations in the reproduction of the original process in the manual.

It is proposed that this mechanism be retained as the logic is strong and the programming in place already within the algorithm to perform the calculations.

To gain an understanding of how the maintenance-reseal cycle lengths affect the total PV amount, a complete understanding of the mathematics is required. This is achieved by analysing the procedure that was used to build the PV section of the TSA. Once this is understood, changes can be made to the TSA by decreasing the economic factor from 10% to a more pertinent 6%.

The calculations give rise to a PV factor based on the cycle length and the remaining life of present surface. This PV factor, when multiplied by the reseal cost, gives an assessment of the whole-of-life costs for that option. The finite value of the factor arises due to the discount factor producing converging infinite series of repeated reseal costs and maintenance cycles.

The corrected PV expression from NZIHT (1999, equation 9) is shown below as equation 5.6:

(Equation 5.6)

$$PV(n,N) = R\left[\frac{g^{N-n}}{g^N-1} + \frac{1}{kq}\frac{g^{N-n}}{g^N-1}\frac{S(N)(kg)^N(g-1)}{k^N(g^N-1)-S(N)g^N(g-1)} + k^{-n}S(n)\frac{(kg)^N(g-1)}{k^N(g^N-1)-S(N)g^N(g-1)}\right]$$

where:

R = reseal cost

g = 1 + discount rate as decimal

N = cycle length, n = remaining life of existing surface before next reseal

S(N) = finite geometric series summing all maintenance costs for N cycle length

S(n) = finite geometric series summing maintenance costs for n years prior to initial reseal

q = 400

k = $q^{1/(N-1)}$

An updated PV factor table was produced for an economic factor of 6%, as this value was deemed more representative of the current economic situation (see table 5.3).

Table 5.3 Present value factor for 6% discount rate

	n →							
N	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**
2	17.7109	16.7109						
3	9.3209	8.8274	8.3290					
4	6.5609	6.2399	5.8936	5.5609				
5	5.1541	4.9276	4.6632	4.4025	4.1541			
6	4.2933	4.1236	3.9123	3.6975	3.4903	3.2933		
7	3.7097	3.5771	3.4031	3.2210	3.0425	2.8717	2.7097	
8	3.2872	3.1802	3.0338	2.8763	2.7196	2.5682	2.4239	2.2872
9	2.9670	2.8785	2.7531	2.6150	2.4754	2.3393	2.2087	2.0846
	1.9670							
10	2.7160	2.6413	2.5324	2.4099	2.2842	2.1605	2.041	1.9269

	n →							
N	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	1.8186	1.7160						
11	2.5141	2.4500	2.3542	2.2446	2.1305	2.017	1.9067	1.8010
	1.7002	1.6046	1.5141					
12	2.3483	2.2925	2.2074	2.1084	2.0041	1.8994	1.797	1.6982
	1.6038	1.5140	1.4288	1.3483				
13	2.2098	2.1606	2.0844	1.9944	1.8985	1.8013	1.7056	1.6129
	1.5239	1.4390	1.3584	1.2820	1.2098			
14	2.0924	2.0487	1.9798	1.8975	1.8088	1.7183	1.6285	1.5410
	1.4567	1.3761	1.2994	1.2266	1.1577	1.0924		
15	1.9919	1.9526	1.8900	1.8142	1.7319	1.6471	1.5625	1.4797
	1.3997	1.3228	1.2495	1.1798	1.1137	1.0511	0.9919	
16	1.9048	1.8693	1.8120	1.7419	1.6652	1.5855	1.5056	1.4270
	1.3506	1.2771	1.2068	1.1398	1.0762	1.0159	0.9588	0.9048

As shown in table 5.3, longer cycle lengths produce a smaller PV factor. When compared with the discount factor of 10%, as shown in table 5.4, it can be seen that a smaller discount rate increases the PV factor as expected.

Table 5.4 Present value factor for 10% discount rate

	n →							
N	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**
2	11.0276	10.0276						
3	5.9104	5.3999	4.9104					
4	4.2241	3.8922	3.5454	3.2241				
5	3.3692	3.1314	2.8620	2.6053	2.3692			
6	2.8478	2.6670	2.4481	2.2327	2.0318	1.8478		
7	2.4955	2.3522	2.1692	1.9833	1.8072	1.6445	1.4955	
8	2.2416	2.1244	1.9681	1.8048	1.6474	1.5004	1.3652	1.2416
9	2.0502	1.9519	1.8161	1.6707	1.5282	1.3937	1.269	1.1547
	1.0502							
10	1.9009	1.8169	1.6974	1.5665	1.4362	1.3118	1.1957	1.0887
	0.9905	0.9009						
11	1.7816	1.7086	1.6021	1.4833	1.3632	1.2474	1.1384	1.0374
	0.9444	0.8593	0.7816					
12	1.6842	1.6199	1.5242	1.4155	1.3041	1.1956	1.0928	0.9968
	0.9082	0.8267	0.7522	0.6842				
13	1.6035	1.5462	1.4594	1.3593	1.2555	1.1534	1.0558	0.9642

	n →							
N	1	2	3	4	5	6	7	8
	9	10	11	12	13	14	15	16
	0.8793	0.8010	0.7291	0.6635	0.6035			
14	1.5357	1.4841	1.4047	1.3121	1.2149	1.1184	1.0255	0.9378
	0.8560	0.7804	0.7109	0.6471	0.5888	0.5357		
15	1.4780	1.4312	1.3582	1.2719	1.1806	1.0891	1.0004	0.9161
	0.8372	0.7640	0.6964	0.6343	0.5774	0.5254	0.4780	
16	1.4285	1.3858	1.3182	1.2376	1.1513	1.0643	0.9794	0.8983
	0.8219	0.7508	0.6849	0.6242	0.5686	0.5176	0.4710	0.4285

It is recommended that the user be able to select the discount factor. This will make it easier in the future to reflect changes in the discount rate policy should the Transport Agency have a shift in policy on this matter. It also allows the user to test different discount factors to assess the sensitivity. This would provide a similar mechanism to varying the BCR in the current setup.

The current process within the TSA is to remain unchanged. There are many different ways of estimating future maintenance costs. **It is recommended** that the current process remain unchanged as it seems to have worked satisfactorily since the inception of the TSA and alteration would require recoding to a new methodology. Any new methodology is unlikely to provide any more compelling argument for accuracy than the current model.

5.6 Step 5: Selection of SCT options

5.6.1 Current TSA process

The SCT options of smoothing and strengthening are assumed to provide similar levels of road roughness after treatment. Thus the option with the lowest total treatment cost plus discounted maintenance cost becomes the preferred SCT option.

5.6.2 Recommended improvement

This step is now redundant as the VOC and BCR methodologies no longer match Transport Agency policies and processes, and strengthening and smoothing treatments are no longer applied.

Also, the BCR method is being replaced by PV analysis. Therefore, there is no assessment of the benefits achieved from the two treatments. The SCT option is replaced by a single option dependent on the assessed mode of failure in the pavement.

5.7 Step 6: Assess the need for resurfacing

5.7.1 Current TSA process

This step decides between the preferred SCT option and the non-SCT alternative chosen in step 2.

If the total treatment cost plus discounted maintenance cost of the SCT option is less than that of the non-SCT alternative, the SCT option is automatically given a higher priority. In other cases the BCR of the SCT is computed.

In the current process, benefits are accrued from reduced roughness levels resulting in lower VOC. If the BCR exceeds a user supplied cut-off value, the preferred SCT option is selected. The benefit-cost value is used as a priority indicator for the list of SCT treatments. **It is recommended** that this process be discontinued as the VOC and BCR methodologies no longer match NZ Transport Agency policies and processes.

It is recommended that the PV be used to assess whether the SCT option is to be selected instead of a resurfacing treatment.

It is recommended that if resurfacing is the selected option, the treatment length should be checked for extreme levels of distress that would indicate a pavement renewal is still required. The basis of this test is detailed in section 4.2. The recommended intervention levels are detailed below in table 5.5.

Table 5.5 Boundary values for intervention

Pavement layer	Parameter	Value
Upper	D_0	>750µm
	RoC	<50m
Subgrade	RoC and $SI_{rutting}$ or rut rate	50m and 4 2mm/year

5.8 Step 7: Resurfacing priority indicator

5.8.1 Current TSA process

If the non-SCT option is selected and this option is for a surfacing in the budget year, a resurfacing priority indicator is calculated.

The additional cost of maintaining the road in good condition for an additional year is estimated by assuming that most of the defects shown in the condition rating will require correction in the budget year and a proportion of these will reoccur and require correction before resurfacing the following year.

This delay cost is divided by the cost of the resurfacing to give the first year rate of return which is used as a priority ranking indicator for the resurfacing list.

5.8.2 Recommended improvement

It is recommended that a maintenance cost test be applied also. **It is recommended** a test for pavement and maintenance costs in the last 12 months prior to three months before the analysis date be added to the delay cost assessment.

The delay cost used in the priority ranking test is the greater of the two costs calculated from correction of current faults and assumption of ongoing future maintenance.

5.9 Step 8: Seal widening need

Consider the need for seal widening (for maintenance reasons rather than safety). The annual rate of deterioration is calculated by dividing the amount of edge break plus edge break repairs by the surfacing age. If the rate exceeds 5% per year, the road is reported for a possible widening.

There are no recommended changes to this mechanism.

5.10 Step 9: Drainage maintenance needs

TSA lists all drainage maintenance costs and requirements. For resurfacing or SCT, all defects are assumed to require rectification.

There are no recommended changes to this mechanism.

5.11 Additional comments

The TSA assumes that the treatment lengths set up in the RAMM database are appropriate. A treatment length can be defined as uniformly performing differently from those either side. A tool to assist with this is the 100m prioritisation tool detailed in NZ Transport Agency (2013) *Manual management plan for state highway annual planning instructions manual*. The 100m sectioning of each treatment length has each portion assessed for a number of faults to check performance is homogenous across the entire length. A special focus should be on treatment lengths that are longer than say 500m. This task could be undertaken on sites recommended for review by the TSA. The risk is for those treatment lengths where a short length should be renewed but the triggers are masked as the remainder of the treatment length is in good condition.

6 Conclusion and recommendations

6.1 Conclusion

The New Zealand TSA compares very well internationally, especially considering it was developed over 25 years ago. One of the advantages of the TSA is the consideration of future surface lifecycles and maintenance costs in the economic assessment of treatment options, usually only seen in more complex predictive type systems. The trigger-based condition flowchart used to determine renewal need is sophisticated and based on a number of parameters. It calculates treatment and maintenance costs and examines drainage and seal widening. It allows a variety of economic scenarios to be tested. The TSA does not predict pavement condition and looks at the short term only. However, its purpose is to identify candidate sites for the short-term FWP, rather than quantifying medium- to long-term needs and such a predictive function is therefore unnecessary.

Its main weakness compared with international practice is the lack of FWD information as an indicator of pavement strength and durability. The use of the BCR as an economic assessment is now outdated but is still more advanced than international practice elsewhere. The algorithm also needs to consider maintenance costs and incorporate thin asphaltic surfacings.

Improvement recommendations therefore focus on improving aspects of the algorithm rather than changing the core process itself. The most significant recommendation is the replacement of the BCR and VOC based benefit with a PV analysis. This also brings a more flexible approach to the use of discount factors. The second major recommendation is to include the use of FWD data, in particular to determine the cause of pavement failure and therefore treatment type. It should be noted, however, that the treatment types recommended in this review are not much different from the current ‘smoothing and strengthening’ options in terms of cost and assumed treatment form.

6.2 Recommendations

The following is a summary of the main recommendations arising from the research project:

- Include historic maintenance costs to test, in addition to the current condition, whether the truer life of the current surfacing has been extended beyond its optimum intervention.
- Add a new cost set table for thin asphaltic surfacings. The trigger for utilising an asphaltic surfacing treatment would operate on a ‘like-for-like’ basis.
- Replace smoothing and strengthening options with a modified basecourse treatment and a full pavement renewal. The treatment type options within the TSA for smoothing and strengthening can be applied as per the basecourse improvement and full pavement renewal respectively.
- Where FWD data is available, use a combination of radius of curvature and central deflection to determine the failure mode and therefore treatment option for pavement renewal.
- Where HSD is available and there is no FWD data, use the flushing test to determine a possible pavement failure and therefore the pavement renewal treatment required.

- Where no FWD or HSDC data is available, apply the following test is applied to determine the nature of the pavement renewal:
  - urban locations
    - <10,000vpd and/or collector or below basecourse improvement
    - >10,000vpd and/or arterial or above pavement improvement
  - rural locations
    - basecourse improvement.
- The TSA process allows for customisation according to road classification. A simple method such as a user-defined table populated with standard default settings for each road classification could be applied.
- Allow the user to select the discount factor.
- Discontinue the BCR determination and use the PV to assess whether the SCT option is to be selected.

It should be noted that there are a number of smaller, more detailed recommendations within the report for improvements to the TSA.

6.3 Further work

This research has shown that adequate data is required to achieve more robust and reliable inferences regarding the possible parameters for use in the improved TSA and the values defining them as decision criteria for pavement rehabilitation needs. Therefore the analysis of sample sizes from other RCAs with a sufficient number of data points for all three data types (FWD, condition and SI) is recommended. Through analysis, the correlations obtained in this research will be confirmed, and other useful correlations may be acquired for use in the improved TSA. In addition, through the strength of the correlations, criteria for the implemented parameters defined as the critical indicators for rehabilitation treatments should be obtainable.

In addition, parameters deemed significant and with critical values indicating the need for rehabilitation as referred to in section 4.2.4, should be tested on a New Zealand road network to confirm their robustness and reliability for implementation into the improved TSA.

There is scope for further investigation as to whether a composite index would provide any additional benefit. The first aspect is quantifying what proportion of treatment lengths would fall into this category and whether the inclusion of the composite index would create a benefit to justify the alteration. COST (2008) details the calculation of composite indices for road pavements and gives advice on how they can be developed. The second aspect is quantifying how many treatment lengths would fall into this category and whether the inclusion of the composite index would create a benefit to justify the alteration.

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Appendix A: Glossary

AASHTO	American Association of the State Highway and Transportation Officials
AMP	asset management plan
BLI	base layer index
BCR	benefit-cost ratio
CCI	cost construction index
COST	European Cooperation in Science and Technology
D0	falling weight deflectomer maximum deflection
dTIMS	Deightons Total Infrastructure Management System
ESA	equivalent standard axle
FWD	falling weight deflectometer
FWP	forward work programme
FYRR	first year rate of return
HSD	high-speed data
IRI	international roughness index
LHS	left hand side
LLI	lower layer index
MCC	Manukau City Council
MLI	mid layer index
MODCALC	a modular arithmetic calculator software package
NAASRA	National Australian Association of State Roading Authorities
NSCC	North Shore City Council
NSH	national strategic highway
NZIHT	New Zealand Institute of Highway Technology
PASER	pavement surfacing evaluation and rating
PII	pavement integrity index
PV	present value
RAMM	Road Asset and Maintenance Management (database)

RAMM Contractor	software by RAMM Software Ltd to collect and update inventory data in the field
RCA	road controlling authority
RCH	regional collector highway
RDH	regional distributor highway
RHS	right hand side
road ID	RAMM road identification number
RoC	radius of curvature
RSH	regional strategic highway
SCI	surface condition index
SCRIM	sideway-force coefficient routine investigation machine
SCT	shape correction treatment
SI	structural index
SIrutting	structural index for rutting
SIroughness	structural index for roughness
SIflexure	structural index for flexure
SN	structural number
SNeff	structural number
TAF	temperature adjustment factor
Transport Agency	New Zealand Transport Agency
TRC	Taranaki Regional Council
TSA	treatment selection algorithm
VOC	vehicle operating cost/s
Vpd	vehicles per day