SUMMARY

A road authority that has pavement assets to manage needs well developed strategies for network maintenance based on desired levels of service, pavement deterioration and budget constraints supported by options for cost effective pavement rehabilitation once a terminal condition is reached. A purely reactive approach to maintenance or rehabilitation can result in both inefficiencies and unexpected costs. This is being further highlighted by the need to consider impacts of the newly increased VDM limits.

In recent years, a wealth of pavement structural data has been collected and stored in RAMM, but it is only used superficially. The bulk of the data is neglected. The mechanistic approach to pavement management uses Austroads principles to analyse the structural data already collected in RAMM, then calculates layer stiffnesses and stresses and strains induced in the pavement structure by trafficking to rationalise and quantify distress modes. This article addresses the role of structural analysis using a mechanistic approach to reduce the cost of maintenance and rehabilitation by effectively identifying the relevant distress mechanism, and focussing treatment only on subsections where work is essential, rather than using a continuous, and sometimes ineffective, treatment over an assumed treatment length. Often only the carriageway interval or top surface type is used for identification of treatment lengths.

In many cases, the existing raw data already stored in the RAMM database can be re-analysed to minimise any further data collection costs. Significant advancements in the application of pavement structural analysis have taken place in 2008 and 2009 as part of Austroads and NZTA research, and these are summarised in this article.

The following aspects are addressed:

- Part I - Network maintenance and determination of Forward Work Programmes
- Part II - Pavement structural rehabilitation
- Part III - Quality assurance of newly constructed pavements and demonstration that the intended design life will be achieved
- Part IV - Maintenance cost implications of increased vehicle weight limits

The conclusions to each of Parts I, II, III & IV highlight the key benefits that can now be obtained from a mechanistic approach to pavement asset management giving cost-effective design, reduced incidence of premature failure, reduced cost for maintenance, and informed budget estimates for future planning.
1 Part I: Forward Work Programme

Generally, a Forward Work Programme (FWP) is a proposed timetable of maintenance (and new construction work) that allows the pavement network to sustain the expected traffic levels while keeping the pavement condition above a certain standard and remaining within the approved budget. To implement a FWP for a pavement network, a Pavement Performance Model (PPM) is needed to predict the rate of deterioration and hence the timing of future interventions.

It is important to stress that all prediction systems are limited by the quality of the input data, and to get meaningful predictions, comprehensive condition data are required. Often FWPs are calculated at a network general level giving a picture of the level of deterioration in the network (without identifying specific roads, using precedent quantities to calibrate the model), as opposed to the network specific level required to predict rehabilitation needs for identified roads or treatment lengths.

This article identifies a limitation of the current FWP system and recommends an improved methodology incorporating pavement structural information, which for many networks is already available in RAMM; namely deflection testing (obtained by FWD).

1.1 HDM

The World Bank has invested heavily in producing generic Highway Design and Maintenance (HDM) Models. Both HDM-III (1) and the subsequent revision, HDM-4 (2) models, develop FWPs based on empirical measures for pavement deterioration. All models require calibration to local conditions. The degree of calibration depends on how different local conditions and materials are to those used in establishing the original HDM models.

1.1.1 Pavement Performance Models

The deterioration models contribute to the derivation of a Pavement Condition Index (PCI) that indicates the relative state of the entire network. For the duration of a FWP, the HDM model seeks to maximise the PCI subject to the budget allowed.

There are three evaluation models used to do this:

a) Trigger Model - where treatments are proposed upon pavement deterioration reaching a certain state. The treatments are proposed subject to the available budget (i.e. prioritised), based on need to maintain certain pavement conditions, or ‘triggers’, within the FWP.

b) Optimal Model - where proposed treatments are allowed to vary to optimise PCI with respect to budget. This allows the network manager to maximise pavement condition over the duration of the FWP.

c) Specific Model - this looks at the impact that specified treatments have on the FWP. Typically, this sort of analysis is done upon an FWP adapted from either the trigger or optimal model after review of other factors.

1.1.2 Level of Consideration

Network general PPM outputs are applicable where the collected database is sparsely populated and treatment lengths are either undefined or there are only sufficient data points in each treatment length to define probable mean values. The output comprises the number
of lane kilometres of each type of rehabilitation required each year for a defined period of years, but does not define where the work is required.

Network specific outputs are applicable where there are sufficient data to reliably propose a treatment for a given treatment length. The output comprises the specific treatment length, location, anticipated distress mode, rehabilitation option and treatment year.

The HDM model provides a manageable way for roading authorities with limited resources to adopt a forward work programme that meets their budget constraints. Results for network general analyses have been very good, and are relatively well calibrated to New Zealand conditions. However, the more detailed pavement deterioration models have proved difficult to calibrate at a network specific level.

1.2 dTIMS

The Deighton Total Infrastructure Management System (dTIMSTM) software, originally incorporated the HDM-III model. This was upgraded in 2004 (dTIMS CT), and incorporates more sophisticated features, as well as many pavement performance models from HDM-IV.

The dTIMS model intended to provide 10 or 20 year FWPs while RAMMs Treatment Selection Procedure provides focus on the next year or two.

The dTIMS models are provided with default coefficients which require calibration for local conditions. The process of level 3 calibration of the HDM pavement performance models (PPM) to local conditions has occasionally resulted in replacement of the HDM model with one derived from local data (3; 4). Part of the problem experienced is the quality of input/calibration data i.e. the national Long Term Pavement Performance sites (LTPP) show minimal rutting or roughness progression as yet.

However, a more significant problem has been the poor predictive power of the empirical parameter (Adjusted Structural Number, SNP) used for modelling structural deterioration of pavements. This concept originated in the ASSHO Road Test in the 1950’s and is now well outdated because there are now reliable procedures for assessing layer stiffnesses, stresses and strains throughout the pavement. A comprehensive discussion of this can be found in (5).

1.2.1 Model Reliability

When initially seeking to determine a FWP using a trigger model, there will typically be treatment sections that already meet or exceed the trigger criteria in the first year. As a consequence there will be an associated backlog of outstanding works. These works that meet trigger criteria but have yet to be conducted are not part of the predictive process and so are not influenced by PPM reliability. They are dependent solely on the suitability of the trigger criteria. As the FWP progresses, the trigger model develops an increasing dependence on the reliability of the PPM.

On the other hand, the optimal model requires much greater dependence on PPM validity and reliability, as it seeks to maximise PCI throughout the FWP. Currently the only measure of structural capacity used in any HDM model is SNP.

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1 See [http://www.ingenium.org.nz/IDS+Ltd.htm](http://www.ingenium.org.nz/IDS+Ltd.htm) or [http://www.deighton.com](http://www.deighton.com) for more details
In the dTIMS PPM, SNP is a key parameter for predicting cracking initiation, rutting, and roughness progression. Therefore, in optimising PCI, the PPM targets specific treatment lengths where SNP is low (because these are inferred to be the most structurally vulnerable points); assuming remediation will result in greatest increase in PCI. This is valid, only to the extent that SNP is a good predictor of pavement deterioration for the relevant distress mode applicable to specific treatment lengths.

To use the optimal model to predict structural remedial work for a FWP, it is necessary to demonstrate, for the specific treatment lengths which are candidates for renewal, that SNP does predict the distress mode (or modes) causing the structural decrease in PCI. Assuming it does, the optimal model should offer good reliability. If not the model may well be responding to a circular argument, and the FWP (apart from any backlog) would be highly suspect.

### 1.2.2 Adjusted Structural Number (SNP)

Field experience shows that, once calibrated, SNP tends to predict overall performance of a network, but there is a lack of convincing evidence that SNP is a valid predictor for any specific treatment length for any distress mode. For example, Henning et al (4), in level 3 calibration of the dTIMS PPM, conducted regression analyses and found that SNP did not predict cracking initiation.

SNP, being related to the load spreading ability of a pavement and hence limiting strains at the top of the subgrade, is more likely to be a predictor of rutting rate rather than any other distress mode. Therefore one measure of SNP’s reliability (or otherwise) is to compare the rate of historical rutting for individual candidate treatment lengths in relation to their current SNP values. If approximate dependence is indicated, then the optimal model for the FWP can be adopted. If not, the trigger model FWP should be considered (particularly if it is consistent with historic quantities of treatment per year).

Figure 1 plots the rate of vertical surface deformation (VSD) per million equivalent standard axles (MESA) against traditional SNP. This is an example of a pavement with a slight trend for increasing deformation rate with decreasing SNP (as might be expected, i.e. VSD rate, and hence rutting rate, is inversely related to SNP).

![Figure 1 - Pavement example where rutting rate is weakly related to SNP](image)

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2 This is problematic, as most NZ RCAs report that rutting is seldom a driver for pavement rehabilitation.
Figure 2, in contrast (a road from the same network), shows absolutely no hint of any trend of rutting dependence.

![Figure 2 - Pavement where rutting rate is totally unrelated to SNP](image)

In most networks examined, results like Figure 2 are typical while Figure 1 results occur rarely. In theory, when the optimal model is verified, it provides a superior model for determining FWP. However in practice, when using SNP, the Trigger Model with nominal optimising of any existing works backlog may prove more reliable.

### 1.2.3 Treatment Length Determination

Treatment lengths are normally obtained from the network database. The stresses and strains in the various layers are not traditionally made available for the HDM/dTIMS models.

## 2 Mechanistic Model

### 2.1 Background

A Forward Work Programme (MFWP) has been developed based on a mechanistic model of structural deterioration for NZ unbound granular pavements with chip seal or thin asphaltic surfacing (6). Structural parameters consistent with Austroads principles for pavement deterioration have been deduced from Accelerated Pavement Testing (APT at CAPTIF) and analysis of Long Term Pavement Performance sites in NZ. Based on this Interim APT-LTPP (IAL) Model, prediction of a more rational Forward Work Programme is possible. Moreover, the IAL Model can be readily improved as further information becomes available - either from pavement performance observations of specific distress modes within a given network, or from the LTPP sites - as opposed to SNP which is severely limited by its attempted “one size fits all” role. For reasons outlined above, the Trigger Model is recommended unless the PPM parameter for structural evaluation is shown to be related to the specific distress mode.

The Mechanistic Forward Work Programme (MFWP) utilises the same surface condition and trafficking data as other FWP systems, but utilises the back analysis of Falling Weight
Deflectometer deflection bowls to determine the approximate stresses and strains induced throughout the pavement when subject to an ESA loading.

### 2.2 IAL Model

The FWD test is a dynamic test that can cheaply and quickly give an indication of pavement structural capacity. Currently this is simply denoted as SNP. However, since FWD data collection has now become standard practice and is commonly recorded in RAMM, no extra data are required. Reprocessing of the data to develop a full structural model is cost effective and allows the network manager to maximise the benefit obtained from the nature of the FWD test (6).

The additional work, beyond traditional determination of SNP, involves analysis of the structural data to ensure a rational multi-layer elastic pavement model is obtained.

The IAL Model, (7), describes pavement remaining life (ESA) over four primary structural distress modes:

- **Rutting** – vertical surface deformation resulting primarily from one dimensional densification (compaction) of the pavement layers and the subgrade. Some lateral movement may also take place in the early life of the pavement but in the current classification for “rutting” it is assumed these lateral movement rates will be minimal after the bedding-in phase.

- **Shear** – lateral deformations or shoving within the pavement layers primarily related to shear. There will be an associated increase in rut depth which is likely to increase rather than stabilise with ongoing load repetitions. Shear instability will commonly lead quickly to cracking of the surfacing, pumping and potholing.

- **Roughness** – loss of shape longitudinally along each wheel path. This is primarily governed by structural non-uniformity (longitudinally) leading to variations in rut depth. Roughness is also a secondary effect of shear instability.

- **Flexure** – the imposition of horizontal strains within the surfacing as a result of trafficking. Strain reversal will occur as the deflection bowl passes along the wheel path; (compressive–tensile–compressive) at the bottom of the surfacing and generally the reverse sequence at the top of the surfacing. The tensile strains eventually result in crack initiation within the surfacing. Additional surfacing may be sufficient for substantial life extension if the existing surfacing is thin (and as yet relatively uncracked). However, thick or aged surfacings suffering from flexure are likely to require replacement or other structural rehabilitation.

In reality, all of these mechanisms for pavement structural deterioration are inter-related. Once cracking has initiated, and there is water ingress, a pavement can develop accelerated rutting, pumping, shallow shear and pot-holing which in turn lead to rapid deterioration of a pavement’s roughness. Correctly predicting the occurrence of this first significant failure mechanism can lead to timely and appropriate intervention and significantly prolonged pavement life.

Adopting the above concept of determining pavement structural life for the four distress modes has its parallels in other forms of structural engineering e.g. the design of a structural column or beam for multiple distress modes (bending moment, buckling, deflection and shear capacity).
2.3 Mechanistic Forward Work Programme

2.3.1 Structural Analysis

To adopt a mechanistic FWP, additional engineering inputs by a pavement analyst are required. These involve preparing the inputs to dTIMS/HDM and checking the validity of the resulting model. This is a matter of a few minutes per kilometre. Structural analyses do require reasonable knowledge of the nature and thickness of any bound layers. Knowledge of the thickness of unbound pavement layers improves accuracy, but in the absence of such information an experienced analyst can reliably model the majority of pavements using the data now stored in RAMM. The minority that do not result in unequivocal models can be flagged for future destructive testing if higher precision is warranted.

Once the structural analysis is completed a set of structural parameters are generated plus a refined mechanistic treatment length table, for input to HDM/dTIMS. The parameters are SNP (for traditional users) and 4 additional structural parameters, one for each of the 4 distress modes listed above. These are either inputted directly to HDM/dTIMS in place of SNP, or used to calibrate regression equations for future performance based on past observed performance. The process simply brings HDM/dTIMS predictions more in line with the widely accepted principles given in the Austroads (2009) Guide to Pavement Technology Series, and removes the otherwise wide divergence in rationale and results.

If there is limited calibration information for the specific network, or no dTIMS/HDM is planned for the network, a FWP focussing only on structural rehabilitation can still be generated using the IAL Model.

2.3.2 Treatment length tables

A treatment length is defined by NZTA as: “a uniformly performing contiguous section of road, which is performing differently from the adjacent sections.”

For rational roughness progression modelling, treatment lengths should also be further separated into lanes, but this is currently not carried out in many systems (unless there is a divided carriageway).

Treatment lengths are required by NZTA to be allocated as follows:

   (i) Initial Selection: physical constraints (e.g. end of a road), change in road width, change in top surface layer, either changes in chip type or other surfacing treatment
   (ii) Refinement: “When it becomes obvious that a seal length is not performing in a uniform manner, the treatment length shall be redefined to restore uniformity within the treatment length.”

Mechanistic analysis is a highly cost-effective tool for achieving the refinement (subsectioning) required. Invariably, mechanistic analysis will lead to additional, yet very relevant, sub-sectioning as the manner in which the pavement behaves structurally becomes revealed. This is a key benefit from the structural analysis as structurally weak sections are clearly differentiated from the sound sections. This allows treatment lengths to be confined to relevant sub-sections rather than have treatment extending collectively over both sound

and unsound sections. An example is provided in the following figures where the X axis is chainage, and various mechanistic parameters are displayed on the Y axes.

**Figure 3 – Pavement where only initial treatment lengths have been applied**

Figure 3 shows a portion of road recently tested where an initial treatment length (defined by the RCA only on the basis of top surface) starts (vertical red line). It is apparent from the data beyond, that there is a great deal of fluctuation across all of the mechanistic parameters. This initial defined treatment length is over 2 kilometres in length. Taken as a single treatment length the Forward Work Programme would ignore several sub-sections within it, that are clearly far from uniform, but this will generally not become evident until the mechanistic analysis parameters are examined. Figure 4 shows the new sub-sections, some of which require no structural treatment, that an analyst would delineate (vertical blue lines) after analysis of FWD results for this road. The 2 km section has been sub-sectioned to an average of 200 m which is still a workable treatment length. The thickness of the treatment (for an overlay option in this instance) is shown as a red line capping the top graph of the set. Reconstruction in this case would be the likely rehabilitation treatment, but the same treatment length limits would result from either solution.

**Figure 4 – Pavement where initial treatment lengths have been supplemented with manual sub-sectioning after mechanistic analysis**
In the above example, the refined sub-sectioning determined from the structural analysis would reduce the 2 km of rehabilitation to about 1.3 km with very substantial cost savings, and hence an improved Benefit-Cost Ratio\(^4\).

### 2.3.3 Level of Service Conditions

Network-wide pavement condition measures are used to trigger rehabilitation. The default values assumed for state highways are from the Transit State Highway Asset Management Manual. These depend primarily on traffic density (AADT). For local authority roading networks, the defaults are given in Land Transport NZ Maintenance Guidelines for Local Roads\(^5\), and these are based primarily on AADT, terrain, %HCV and road type (Rural / Urban).

### 2.3.4 Distress Mode Targets

It is useful to explore calibration of a preliminarily generated FWP for a network under consideration using precedent. One measure of this is the relevant proportions of the four different structural distress modes experienced historically. This may come from records of past experience, or by systematic survey of the currently exhibited distress modes. However, any calibration factors resulting that differ from the national average will need to be reviewed critically.

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\(^4\) Given pavement rehabilitation costs, in the urban area from which this case history came, in the order of $1,000,000 for 2 km, this indicates a cost saving of $600,000 for expenditure of less than 0.2% of the saving, for FWD testing inclusive of mechanistic analysis.

3 Conclusions: FWP

- The Mechanistic Forward Work Programme is intended to be supplementary to existing dTIMS/HDM systems. It addresses only structural deterioration, but in four separate modes of distress (rutting, roughness, flexure and shear) by consideration of the stresses and strains in the various pavement layers and subgrade when subjected to traffic (1 ESA load).

- Other non-structural issues, such as reseals and direct replacement of fatigued asphalt with the same thickness of new asphalt, are not currently included in the MFWP because the issues are effectively addressed in the current system.

- Rational refinement (sub-sectioning) of treatment lengths in accordance with NZTA policy is a primary cost-efficiency that results from mechanistic analysis, allowing targeted rehabilitation only where it is necessary.

- The same applies to RAMM’s Treatment Selection Procedure which would benefit substantially from structural sub-sectioning of treatment lengths as well as more effective inclusion of the wealth of pavement structural data now being stored in RAMM.

- Although Adjusted Structural Number (SNP) is effective as a general indicator of average network wide structural capacity, it is a poor predictor of pavement deterioration under a specific distress mode for any given treatment length. After calibration SNP will therefore give a prediction of what total length of rehabilitation will be required in the short term. However, it has very low reliability for identifying which specific roads will require treatment, the sub-sectioned limits of required treatment, its nature, or its timing.

- Unless SNP is shown to be related to rutting rates, its use in any network specific optimal model based FWP may well be meaningless. Hence caution should be adopted when using this method for assessing medium or longer term funding for a network.

- Existing structural condition data stored in RAMM can be readily re-analysed using mechanistic concepts. The structural data can be relatively old, and even if rehabilitation has been carried out, approximate methods are available to minimise or avoid the need for data recollection.

- After calibration, the MFWP provides a mechanistically sound prediction of the structural deterioration of the network. The MFWP can be readily integrated with other HDM/dTIMS FWP projections for non-structural (maintenance) activities when used as a Specified Model FWP within the HDM/dTIMS model.
4 Part II: Pavement Rehabilitation

4.1 Introduction

Background to a pavement performance model based on Accelerated Pavement Testing (namely CAPTIF) and the national LTPP sites is given in Section 2, above. This model has now been applied to a number of roads, and by applying a feed-back loop, the model can be either validated or calibrated to local conditions.

This section describes the structural information that can be obtained from a pavement in terminal condition, both for the purposes of allowing informed structural design of rehabilitation (by understanding the mechanism of existing distress) and to promote the extraction of information for calibration of a pavement performance model from pavements with known history.

In some cases there may be no information regarding structural maintenance or past traffic, in which case the model provides a prediction of the future traffic that can be expected if the road is subject to smoothing only, ie no structural treatment.

4.2 Structural Evaluation of Pavements in a Terminal Condition

Figure 5 shows a standard structural analysis of FWD data displaying chainage along the x axis and a variety of parameters on the y axes. These include the layer structure, basic deflection parameters, and moduli of the layers. Also indicated are at least 3 options for rehabilitation treatment (granular or asphaltic concrete overlay, cement stabilisation or foamed bitumen stabilisation). Further details of these parameters are given in Appendix A.

Correct assessment of the distress mode (ie the reason a pavement has reach a terminal condition) is required in order to assess whether simple smoothing is sufficient, or whether the structural capacity needs to be increased (by overlay or reconstruction that increases the load carrying capacity). A simple guide to the distress mode is readily generated from deflection data. Figure 6 shows the total expected life for the same pavement analysed in Figure 5. For each distress mode, the total lifetime traffic (Equivalent Standard Axles) is shown using the IAL model prediction.

If the road concerned is not in a terminal condition but its current condition (in terms of rutting, roughness and cracking history) is available, then the remaining life could also be calculated. Figure 6a gives the lifetime ESA for each distress mode with respect to chainage along the road, while Figure 6b gives the same information in the form of a cumulative distribution. In Figure 6b, the governing life is also shown, ie the percentage of road in a terminal condition taken over all distress modes.

For new construction, NZTA policy is to require that 95% of the road should last for the intended design life, ie only 5% should reach the terminal condition. In practice, particularly with local roads, rehabilitation may not be in invoked until at least 10% (and possibly more like 50%) of the treatment length has reached a terminally distressed condition. To demonstrate the measure, the 10 percentile life has been identified in Figure 6b. Using this measure, this case history as an expected lifetime ESA of slightly over 2 million ESA, and is limited mainly by flexure (cracking) but if this could have been controlled by better design, a life of 5 million ESA would have been expected until roughness alone became critical (comparing the dashed and red plots at 10% on the cumulative distribution, Figure 6b).
Figure 5 – Pavement structural analysis
Figure 6a and 6b – Total life for each distress mode and governing life
By comparing the observed performance with the predicted performance, Figure 6 provides a convenient means of assessing:

(i) At project level, the relative performance of different parts of the road or treatment length under consideration (looking at each distress mode separately) which facilitates understanding of critical distress modes experienced and hence informed design of the effective and targeted form of rehabilitation

(ii) At network level, Figure 6 allows the validity of the modelled Forward Work Programme to be determined, and recalibration can be carried out if necessary using relevant performance in the locality.

Calibration of the rutting and roughness progression models is greatly facilitated if both rutting and roughness high speed data are available and the past ESA is known. For roads that are mature but not in a terminal condition, future condition for these two distress modes can be reliably predicted.

The flexure model is much less robust as there is no convenient condition information captured by the high speed surveys. Consequently, the graphs shown may require a general shift up or down, from the position shown. However the relative performance (laterally) along the road should be reasonable for the flexure model, ie the low points should be the locations that fail first and, when they do, an appropriate vertical shift to the full graph can be applied.

The shear instability model is presently the most difficult to calibrate but this can be improved if the laboratory parameters for the basecourse (M/4 suite of tests) and subbase are available as well as the full deflection-time history for deflection testing.

Once the observed (actual) performance is compared with the predicted behaviour, a calibration factor can be readily generated for re-analysis of the specific road. If a number of roads all show similar calibration factors, the calibration factor may then be adopted for the network. If different roads show different calibration factors then the fundamental basis of the performance model should be re-appraised.

5 Conclusions: Rehabilitation

The structural analysis and predicted life graphs for pavements in a terminal condition provide an effective means of addressing the following:

- Sub-sectioning of treatment lengths for focussed rehabilitation and savings from the distinction of those sub-sections which required only smoothing rather than more costly structural rehabilitation.

- Determination of the distress mechanism allowing (i) informed design of cost-effective structural rehabilitation and (ii) avoidance of a repetition if failure has been premature

Where past history (ESA) is known, each treatment length that has reached a terminal condition can be used to further calibrate the Pavement Performance Model so that greater reliability and longer time analysis period can be obtained for Forward Work Programmes of other pavements in that locality.
6 Part III: Quality Assurance

6.1 Design Verification - Demonstration that the intended pavement design life will be achieved.

The future performance (expected life) of a new or rehabilitated pavement is most practically verified by non-destructive methods (eg deflection tests), in conjunction with as-built information. NZTA’s requirement is typically that 95% of the pavement should not reach a terminal condition until at least 25 years of traffic has been sustained. A readily appreciated way of demonstrating life is with cumulative distribution curves for pavement parameters and noting the 95 percentile values.

Until recently the only measure in common use practical was to determine if the deflection of the pavement complied with the empirical Austroads allowable values. However, because each potential distress mechanism is not considered, the reliability of predictions has been limited.

The APT-LTPP Model discussed in Parts I and II, provides a much more fundamental approach in accordance with widely accepted Austroads principles, ensuring that a check of future pavement life covers each of the four structural distress modes (rutting, roughness, cracking and shoving).

The life prediction process can be applied to new full depth construction or newly rehabilitated pavements. Ideally bedding in should be completed (ie traffic of at least 10,000 ESA should be applied). However, quality assurance testing before sealing can also be carried out so that any necessary intervention can be carried out before surfacing is applied. A third application is where a change of ownership of an existing road is intended and an assurance is sought for a minimum life without the need for any structural maintenance.

6.2 Application

The application of the method uses the same principles described in Part II but builds on calibrations obtained from terminal pavements elsewhere and, where practicable, from the same locality. For a new pavement the total life is calculated, while for those with some trafficking already, the remaining life is calculated. In this manner a test of both under and over-design is available.

A case history of an existing pavement where remaining life required determination is shown overleaf in Figure 7. In this case, because minor distress had developed, the high speed data for rut depth and roughness was collected first and incorporated into the remaining life calculation. The figures demonstrate that the remaining pavement life (assuming a 10% cut-off) is predominantly governed by flexure (cracking initiation) and the road is assessed to have a remaining life of 300,000 ESA given its current condition. If cracking could be controlled by frequent reseals (not usually economic), it would be possible to extend the life to over 1 million ESA before both rutting and roughness would also become terminal and rehabilitation would be necessary. A targeted programme of AC overlays might also be considered as this could be used to extend the life for all three primary failure mechanisms.
Figure 7a and 7b – Remaining life for each distress mode and governing life
Conclusions: Quality Assurance

- A pavement structural evaluation can readily demonstrate whether a given pavement is likely to meet its intended design life.

- For a recently constructed pavement, structural evaluation will also differentiate if any deficiency is due to design as opposed to a defect in construction practice or materials.

- A key performance measure of either under-design or over-design is obtained, opening opportunities for refinement of procedures for more realistic design and frequently an appreciation of where cost savings can be made.

- If the evaluation is carried out prior to sealing, timely intervention can avoid the cost and other issues associated with a premature failure.
8 Part IV: Maintenance Cost Implications of Increased Vehicle Weight (VDM) Limits

8.1 Loading Power Laws

Proposed increases in vehicle limits will change the rate of wear of pavements depending on the change in number of standard axles that will be incurred on a given road section.

Section 2.2 describes the 4 structural distress modes than can be evaluated through mechanistic analysis of the pavement. Each distress mode has a different “load factor” or power law exponent, ie a 10 % increase in axle weight will in general lead to an exponential increase in all 4 forms of pavement wear (much greater than 10%). The most sensitive mode to increased VDM is flexure (cracking). The exponents for subgrade rutting, base rutting and roughness are lower than those for flexural cracking of chipseals and asphaltic concrete, which in turn are lower than that for cracking of cemented bound aggregate for which a commonly accepted exponent is 12. (For cracking, identification prior to initiation of distress and early intervention for surface strengthening will be particularly cost effective.)

8.2 Net Present Value Approach

The most pragmatic way to assess the actual road maintenance cost of allowing either increased numbers of heavy vehicles or increased axle loads is to apply a net present value (NPV) approach. The vital first step is to determine treatment lengths rationally from structural analysis (Section 2.3). Then, the Forward Work Programme (FWP) can be assessed for the status quo or “standard” traffic scenario, and again for the increased loading scenario.

The full output for each FWP is a large spreadsheet (not included here) which gives the year in which each treatment length needs to be rehabilitated, the distress mode, and type of treatment required (normally running to at least 10 years but runs of 25 years or more take minimal extra effort). Accuracy diminishes progressively with time, but when the same assumptions are applied to two comparative loading scenarios, the net difference in cost still has reasonable reliability for informed decision making.

The following example is for a roading network which was to be subjected to increased traffic from a new quarrying operation. The following figures are a summary of the full FWP and show just the year, type of rehabilitation, cost and number of lane kilometres experiencing a terminal condition from given distress modes.

This example also illustrates the observation that for most roads, the effect of increased loading does not make a large difference to amount of maintenance done in the long term, but rather the rehabilitation is brought forward and it is the time cost of that earlier expenditure that substantially affects the NPV calculation. Similar findings are reported by other RCA’s (10). Present value is calculated using present day construction costs and the discount rate established by the client. The discount rate (which does not include inflation) can be contentious and in this roading context results are highly dependent on the rate prescribed.
Figure 8a - Distress Modes and Annual Maintenance (Standard Traffic on top, Standard Traffic + Increased Loading on bottom)

Figure 8b - Net Present Value calculation over successive years
9 Conclusions: Increased VDM Limits

• It is important that the impacts of increased VDM limits and their effects on pavement maintenance costs are well understood. The current modelling is limited in its ability to provide this.

• Establishing maintenance costs under increased traffic first requires structural delineation of uniform treatment lengths. This cannot be done in a meaningful way without first examining available structural data.

• In recent years a wealth of pavement structural data has been collected and stored in RAMM but is used only superficially by many RCA’s. The bulk of the data is neglected. This degrades the value of FWP’s predicted using RAMM TSA or dTIMS. The status quo (concepts dating back to the 1950’s) has remained unchallenged regardless of major advances in pavement technology, that have been well publicised in the Austroads (2009) Guide to Pavement Technology Series.

• The mechanistic approach uses the all the structural data to calculate layer stiffnesses, as well as stresses and strains induced in the pavement structure by trafficking, to rationalise and quantify distress modes.

• By using all RAMM’s structural data, Forward Work Programmes can be rationally compared, in line with Austroads principles (9), for alternative loading scenarios, in particular for changes in VDM limits.

• Strategies for programming early surface layer strengthening of critical sections of pavements subject to increased VDM limits will greatly reduce maintenance costs.

• The Net Present Value concept applied to respective FWP’s provides an equitable means of evaluating (and assigning) costs to road users for changes in road use.
References


10. Queensland Department of Main Roads (2006). Guidelines for Assessment of Road Impacts of Development

Mechanistic Analysis Information Website www.PavementAnalysis.com
Appendix A

Parameters from Pavement Structural Evaluation

Back analysis of deflection bowls generates a range of structural parameters for the test point under consideration (Figure 5). Some of the more useful of these are described:

Standard Deflection – The central deflection of the FWD bowl standardised to a 40 kN load (approximately equivalent to a Benkelman Beam Reading).

Curvature – The difference in deflection between the standard deflection of the central geophone and the geophone at 200 mm offset.

Normalised Curvature - Curvature divided by Curvature required for that design traffic load using the Austroads Guide for asphaltic concrete pavements. Values over 1.0 would be expected to fail prematurely.

Layer moduli - The stiffness (Young’s modulus) of each individual layer in MPa. In a subgrade the modulus is approximated by 10 times the CBR. In the evaluation graphs the colour of the modulus plot corresponds with the layer with the same colour in the pavement profile at the foot of the page.

Depths for rehabilitation - these are preliminary estimates based on Austroads principles for the relevant depths in millimetres of various rehabilitation options and include Granular Overlay Depth, Asphaltic Concrete Overlay Depth, Depth of Cement Stabilisation (bound layer), Depth of Foamed Bitumen Stabilisation and Reconstruction Depth (assumed all layers above the subgrade are removed for reconstruction or widening).

Structural Index - these are indices that are equivalent to SNP but there is one for each of four distress modes (rutting, roughness, cracking and shoving). The Structural Index (Rutting) corresponds approximately with HDM’s Adjusted Structural Number (SNP).

Subgrade Modulus Exponent – this is an exponent describing the non-linearity of the stress-strain curve for the subgrade. Values of 0 are linear elastic, grading to highly non-linear at -0.6. Low values are often associated with a drainage problem.

Subgrade Strain Ratio – the strain in the subgrade divided by the Austroads allowable subgrade strain for the intended lifetime ESA. Values greater than 1 indicate the pavement is too thin and will not last the intended life. Values just less than one are ideal, but if as low as 0.5, significant over-design is indicated and substantial cost savings should be achieved if design procedures are refined.