SARF
better roads

ROAD PAVEMENT REHABILITATION

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1 INTRODUCTION

1.1 Scope

These notes are intended and will provide practical guidelines for those involved in the pavement rehabilitation of **surfaced** roads. It is not intended as a manual on routine road maintenance but for larger rehabilitation projects requiring more sophisticated inputs and design considerations. Construction would also be on a larger scale, most often requiring the use of specialist contractors. Un-surfaced or gravel roads are not covered in these notes as they require special but more routine maintenance work than surfaced roads. Although pavement design issues are also very basically dealt with this is not intended to be a pavement design course. The South African Road Federation as well as other specialist organisations has special pavement design courses that deal with that fairly complicated issue in more detail.

1.2 Background

As the road infrastructure of South Africa is becoming older more surfaced roads are approaching the end of their design life and need to be rehabilitated to, at least, preserve part of the investment made in the infrastructure and to provide the road user with an acceptable road network free from hazards. It is accepted that routine road maintenance would be regularly carried out to protect the road and associated road elements from premature deterioration and failure. It is, however, so that even with the best routine maintenance the road pavement and other elements would deteriorate or be damaged in time due to a variety of factors of which the following are the most important:

Traffic loading associated deterioration:

Any road is designed to carry a specific number of standard vehicle axles in its design life, being it 20, 30 or whatever years. Many roads do last their design life and more but overloading and an unplanned increase in traffic loading could decrease the design life considerably and may lead to premature failure with the result that large scale rehabilitation of the road has to be done. Many of our roads have reached their design life and would, some or other time, require proper rehabilitation to increase its service life.

Routine maintenance strategy.

Most roads would require routine maintenance and timely resurfacing to protect the pavement structure from premature failure. The road agencies routine maintenance strategy and availability of fund would determine this action and the frequency of interaction. If it is continuously postponed or neglected, the life of the road could be dramatically reduced.

Environmental factors

Prolonged draughts or sudden flooding of a road may cause permanent damage to the road pavement or other elements that could negatively influence the functional or structural life of the road. In addition, with the high temperatures and fairly low humidity experienced in most parts of Southern Africa, asphalt and sealed surfaces may age pre-maturely, causing cracks with associated water ingress of the pavement structure. If these are not regularly sealed, premature failure of the pavement structure may result.

Geometric upgrading for higher traffic volumes

Some of our older roads have much lower geometric standards than is required by the increased traffic volumes. It would, therefore, be necessary to increase the capacity by widening, constructing shoulders and other geometric enhancements.

1.3 Course objectives

This course gives an overview of strategies and procedures to be followed in road pavement rehabilitation. It is not the intension to be a comprehensive pavement design manual as this is a rather complicated and extensive subject on its own. It will, however, endeavour to point the participant in the right direction. References are given for use by the participant of existing more sophisticated procedures currently in use in South Africa. The participant is encouraged to make use of these references and expertise currently specialising in certain aspects of pavement engineering.

The main course objectives are:

- Pavement condition assessment and an introduction to Pavement Management Systems
- Initial pavement assessment
- Detailed pavement assessment
- Selection of maintenance or rehabilitation options
- Practical considerations.
- Materials designs
- Construction tips.
- Operational Health and Safety and Environmental issues.

2 PAVEMENT CONDITION ASSESSMENT

2.1 Regular assessment (Network level)

With regular pavement assessment is meant the yearly or regular evaluation of the pavement condition to determine the functional and structural condition of the pavement. This is usually done on a **network level** and is regarded as an early warning system to enable a road authority to make timely provision for road rehabilitation needs. Some road authorities make use of a planned reseal programme that would imply that a section of road should be resealed after, say, seven years or whatever after the previous reseal to try and prevent further damage. This, however, is not regarded as an intelligent way of deciding on a rehabilitation strategy unless it is merely used as a way of budgeting for rehabilitation actions. The best way to do this regular assessment is by having some functional and structural evaluation tests done on a predetermined regular basis, being it annually, bi-annually or whatever would best serve the interest of the road authority. These tests could then be incorporated into a pavement management system where it can serve as inputs to an evaluation system that will prioritize rehabilitation actions within the budgetary constraints existing within the specific road authority or owner.

The following aspects should form part of the regular assessment:

2.1.1 Functional assessment.

The functionality of a road include those aspects that influence the rideability of a vehicle as experienced by the road user and includes the following:

Riding quality or road roughness.
Rutting
Skid resistance
Visual condition.
Road capacity.

These aspects are shortly described below.

Riding quality (Road roughness):

Road roughness can be described in various ways. The American Society of Testing and Materials (ASTM) defines it as:

"The deviation of a pavement surface form the true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage, for example, longitudinal profile, transverse profile and cross slope."

Riding quality or road roughness is a measure of the longitudinal profile of a road pavement that will impart a feeling of unevenness or roughness to the road user. To make sense out of a measurement of roughness, it must be quantified as a deviation in longitudinal elevation over an interval. The longitudinal profile can be measured in various ways. To be used as a measure of riding quality, it is, however, not necessary to know the exact profile based on static data. It must also be realised that the specific longitudinal profile of a road measured in different lateral positions would invariably differ. It is, therefore, usually measured in the two wheel paths of a traffic lane. Measuring the road profile is, however, not enough. To make the data standard and useful as a measure of road roughness, it is necessary to manipulate the data in a computer program that filters it and calculate a standardised roughness index that

can be evaluated. Usually the roughness data is expressed in terms of the International Roughness Index (IRI). The IRI is usually measured and calculated separately for each wheel track and is averaged for road sections of 100m and usually reported as an average for road segments of 5km. There are various types of profile measuring apparatus that can be used for measuring road roughness. In South Africa, a calibrated laser based profile measuring vehicle (Photo2.1) is normally used. More information on this subject can be obtained from reference 1.
Road roughness is used by some road authorities and also in the HDM4 model to predict pavement life and rehabilitation costs as well as vehicle operating costs as these can, in most instances, be related to road roughness.

PHOTO 2.1 Road surface profiler.

Rutting

The rutting of a surfaced road, where its functionality is concerned, is obvious. Excessive rutting may cause vehicles to follow the rutted track and may become unstable when trying to go out of the rut. It may also cause rain water to collect in the rut resulting in aquaplaning. Rutting may also be an indication of road pavement deterioration and will be further discussed under the structural aspects.

The degree of rutting is determined by measuring the transverse profile of a road. It can be done by simply measuring the transverse deformation using a 2 or 3m straightedge or by using the same high speed profilometer being used for measuring the longitudinal road profile (Fig 2.1). Rutting normally takes place in the wheel paths and the inner and outer wheel paths are separately measured and averaged over 10m. The results are usually also reported as the average rut over 1km.

Skid resistance

The skid resistance of a road is important for obvious functional reasons. Skid accidents often happen and can be prevented by providing a road with a proper skid resistant surface. Skid resistance has two elements namely the fine sandpaper like effect that provides skid resistance on dry roads and, secondly, the courser type of surface that provides skid resistance by allowing water to drain out below the tyre on wet roads and, in all conditions, causes hysteresis of the tyre that absorbs energy and thus tends to slow it down.

Unfortunately, in South Africa, no acceptable method for measuring skid resistance is currently acceptable to all road authorities. World wide various skid measuring apparatus are being used but inter-correlation is not good and has not been established. As an alternative, the texture depth of the road surface that can also be determined by the high speed profilometer can be used as an indication of the skid resistant properties of the road. It is called the "Mean Profile Depth (MPD)". No correlation with any known skid resistance test has, however, been done and it is merely used as an indicator test.

Visual condition survey.

A visual survey of the road is done in accordance with TMH9 issued by the Department of Transport (2). The following aspects are visually assessed and rated as essential or desirable:

	Essential	Desirable
Surface assessment:		
Texture		X
Voids		X
Surface failure		X
Surface cracks		X
Aggregate loss	X	
Binder condition	X	
Bleeding/flushing	X	
Structural assessment:		
Block cracks	X	
Crocodile cracks	X	
Longitudinal cracks	X	
Transverse cracks	X	
Pumping	X	
Rutting	X	
Undulations/settlement		X
Patching	X	
Failures/Potholes	X	
Functional assessment:		
Riding quality		X*
Skid resistance		X*
Surface drainage		X
Shoulder condition	X	
Edge breaks	X	

Each of these aspects are visually assessed on a scale of 1 to 5 in terms of their degree (severity), with 1 being slight and 5 severe, and extent with 1 as an isolated occurrence and 5 extensive occurrence. The assessment segment lengths are 5km for rural areas and 2km for urban roads.

From this visual inspection a visual condition index (VCI) and reseal index as well as other visual indicators can be calculated.

Road capacity.

Although this function is usually separated from pavement engineering, it is an aspect that also has to be considered when road rehabilitation is envisaged. It would not help if a dysfunctional or structurally inadequate pavement is rehabilitated while the road capacity is such that it just cannot carry the current traffic volumes and is geometrically due for upgrading. In such an instance it must be considered to widen the road, add more lanes, build on shoulders, etc. simultaneous with the structural upgrading. This could change the total design approach. This issue will not be further discussed in this course although some of the practical considerations involved in geometric upgrading will be dealt with.

2.1.2 Structural assessment.

The structural condition assessment on a network level is done to give an indication of the structural integrity of the road pavement. Aspects that are used for this are;

Rutting measurements
Visual condition survey.
Deflection survey

The first two factors also used for the functional evaluation of a road surface are useful in indicating possible structural problems in a pavement.
The visual condition survey involves the evaluation of cracking patterns, pumping, settlements, patching, failures and potholes that can be used to evaluate where pavement problems exist as well as the extend thereof.

Rutting will also indicate a problem in the pavement structure. A low degree of rutting may be due to shear displacement in the surfacing and a large degree of rutting may indicate shear displacement in one or more of the pavement layers, most often in the subgrade.

But, for a proper initial evaluation of the structural condition of a pavement, deflection tests need to be done. For that purpose the Falling Weight Deflectometer is usually used although other means of measurement can also be employed.
On a network evaluation level, FWD tests are usually done every 200m in the left hand wheel track of the left lane of the road.
The FWD results can then be used to determine the structural integrity of the pavement by calculating various indices as preferred by the specific road authority. This will be discussed in more detail when the project level assessment of pavements are discussed.

2.1.3 Pavement construction and materials data.

For any structural pavement analysis and evaluation to be made with confidence the pavement materials properties need to be known. That includes pavement layer properties such as a materials classification and layer thicknesses, for example, 150mm G1 base, 150mm C3 subbase, 250mm G7 selected subgrade, etc. (See Appendix A)

These properties can be obtained from the materials as-built data if available. Else, it can be determined from tests pits made during an investigation. For the regular assessment of the pavement condition, this data would not necessarily be available and may make the structural assessment where FWD results are used, less accurate.

2.1.4 Traffic data and road classification

Traffic data is required to be able to estimate future pavement life from the structural pavement assessment. This data should preferably contain the following information.

Road category, i.e. an indication of the design standard of the specific road section as defined in TRH4. [(4)]
Average number of equivalent 80kN axles (E80) per lane per day.
Average number of axles per lane per day.
Expected growth rate.
Lane split for multi-lane freeways.

If this data is not available, the following can be used:

Average number of light vehicles per lane per day
Average number of heavy vehicles per lane per day
Average E80 per heavy vehicle.
Expected growth rate.
Lane split for multi-lane freeways

2.1.5 Cost of rehabilitation actions.

To be able to determine the cost of rehabilitating a specific section of road as determined by the analysis of pavement and traffic data, the cost of each rehabilitation option must be known and made part of the input database. As an example; if the analysis indicates that a single seal only is required to improve the road pavement to and acceptable standard, the cost of such a seal including aspects such as estimated P&G's, typical traffic accommodation costs and line marking must be known. Similarly for all other possible rehabilitation actions. Most road authorities do have an extensive database of old projects from which these typical costs can be determined. If not, the larger Provincial roads departments or SANRAL would be able to provide typical costs.

2.1.6 Budgetary constraints.

Before any rehabilitation project can be planned, it is necessary to know exactly how much money is available for road rehabilitation or has been allocated for that specific road, province, and district. etc.

2.1.7 Standards.

The specific road authority should be able to define the standards for each of the test variables of the various categories of roads in its network. Examples of such standards would be the terminal values for riding quality, rutting, skid resistance, etc. and would be different for the different categories of roads in the network.

2.2 Pavement management systems. (PMS)

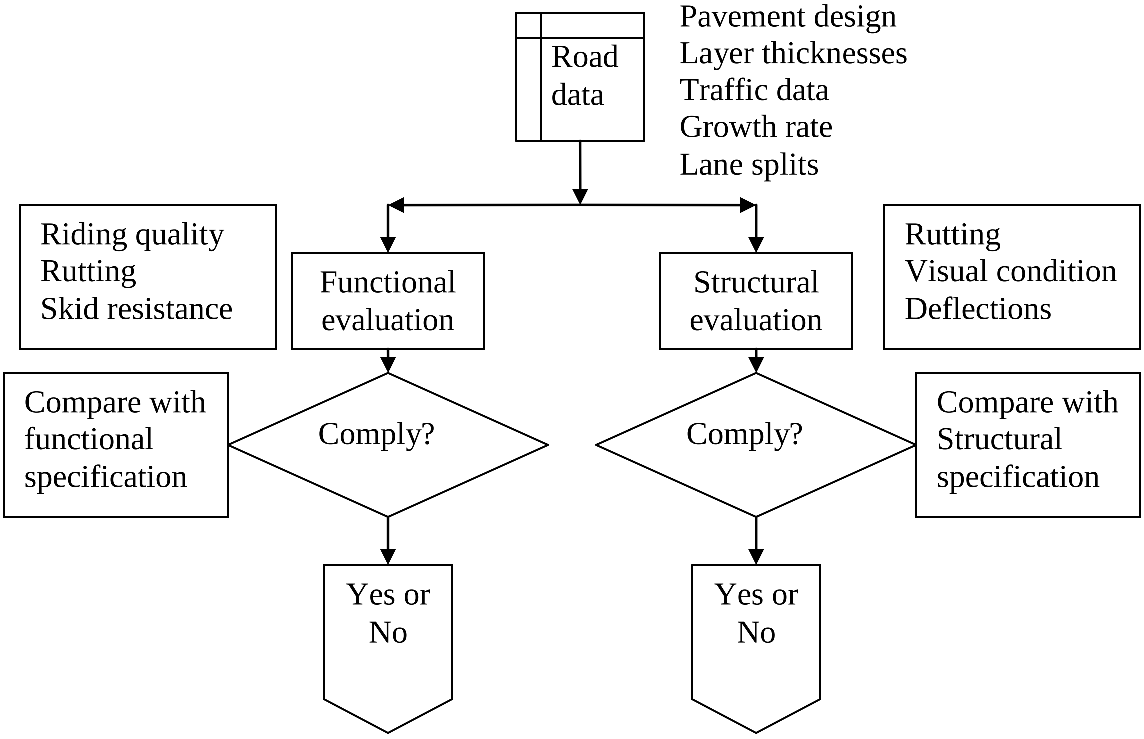
Pavement management systems are usually computer based data collection and manipulation software that is used to store and manipulate the data collected during the pavement condition survey. It usually contains the pavement materials data information for each section of road. During the regular assessment of the network, the pavement condition data is updated and used to evaluate pavement conditions for rehabilitation purposes.

Many different PMS's does exist. The system being proposed by the World Bank is called HDM4, Highway Development and Management System version 4 (5). This series of computer programs using a database for all the data inputs is used for the analysis, planning, management and appraisal of road maintenance improvements and investment decisions. It is a comprehensive system and may also be used to plan upgrading projects etc. This type of PMS is useful in storing and manipulating data and for the preparation of reports and the determination of priorities based on the riding quality, etc. of roads. The basic problem is, however, that the pavement cannot be structurally analysed in such a system. For low volume roads, gravel roads, etc. the riding quality and deterioration in riding quality with time can be an indicator of pavement deterioration and can be related to road user cost.

For fairly well maintained road networks where the riding quality is usually fair to good, future rehabilitation options should, however, depend on the remaining structural life of the road pavements. For that reason one should consider a PMS that will also take that in consideration.

An example of such a system is the PMS developed by Modelling and Analysis Systems (MAS) whom are also the developers of Rubicon Toolbox®, the well known pavement design software package that is being used extensively in the RSA. In their PMS system, a fuzzy logic approach is followed to obtain an estimation of the life expectancy of the pavements. From that they can identify sections of road that are in need of immediate rehabilitation and also that would require interaction in the future.

Figure 2.1 Flow diagram of a pavement management system



OUTPUT:

Option 1:	Functional Yes and Structural Yes:	Do nothing
Option2:	Functional No and Structural Yes:	Re-seal or re-surface
Option3:	Functional No and structural basically Yes:	With slight rehabilitation.
Option 4:	Structural No:	Heavier Rehabilitation.

In a PMS the output recommendations are now listed and **prioritised.** In prioritising the projects, aspects such as road category, traffic and strategic importance should also be taken into consideration.

The estimated cost of the projects are then compared with the budget for road rehabilitation to decide what projects can be funded during a specific financial year.

2.3 Initial assessment (Network level)

During the Network Level assessment of the roads, projects are prioritised and identified for rehabilitation. The network level assessment will also determine the type of rehabilitation that needs to be done for each section of road on the priority list. If a section of road is earmarked for seal only, that action can be performed without any further project level assessment except for the tests required to perform a seal design.
If, however, the network assessment indicated more heavy rehabilitation to be performed such as the reconstruction of the whole or portions of any identified section of road, further assessments and analysis have to be done to be able to determine the extent and detail of the indicated rehabilitation measures.

The steps that are followed in a PMS where the structural evaluation is included are:

- Synthesize surfacing and pavement structural information to obtain uniform segments.
- Extract deflection and condition data from data base and calculate statistics for each segment.
- Perform back-calculation of deflection data to determine the effective stiffness for each pavement layer.
- Simulate pavement deterioration for current year.
- Apply appropriate interventions, if required and re- calculate for each work section.
- Calculate ranking parameters for network (prioritize work sections).
- Sort work sections in order of increasing rank.
- Work down sorted work sections and add rehabilitation costs until budget limit is reached.

F**rom Figure 2.1:**
OPTION 1: Do Nothing:
If the network analysis of the data indicates that a section of road is generally acceptable, both functionally and structurally, nothing further needs to be done and the section is skipped for attention till the next round of assessment tests.

OPTION 2: Re-seal or re-surface:
If the structural evaluation indicates that the road pavement is largely in an acceptable structural condition, only rehabilitation options have to be considered that will improve the functionality of the road. When the riding quality is bad due to surface deformation, an asphalt overlay may be considered instead of a seal. The following table can be used to decide on a rehabilitation option:

Functional failure mode:	**Rehabilitation options to be considered.**
Riding quality:	Asphalt overlay Slurry pre-treatment to better riding quality plus resurfacing.
Rutting:	Rut filling by asphalt, course slurry or quickset slurry (Ralumac or similar). Resurface.
Rutting with pushing out of asphalt:	Milling of rutted areas and replace with asphalt.
Skid resistance:	Resurface.
Visual aspects such as pot-holes etc.	Resurface.
Visual aspects indicating asphalt fatigue:	Resurface or mill and replace asphalt
Visual aspects: cracking.	Seal cracks and resurface.

The resurfacing decision between a seal and asphalt overlay will depend on the class of road, traffic, funds available and the policy of the specific road authority.

The materials properties of the seal and asphalt will be discussed in Paragraph 6.

OPTION 3: Reseal or re-surface with slight rehabilitation.

On most roads one would find limited areas of surface failure or even base failure while most of the road section is still in a structurally good condition. In such a case these areas would identify themselves by showing larger rutting than the rest or more cracking or other forms of failure or fatigue. It would usually be sufficient to visually identify these areas or make use of the rut depth measurements, deflection results and visual inspection to help identify these areas. They can then be repaired by milling and replacing of the surfacing, base or whatever layer has been found to have failed. All cracks must be sealed before resurfacing. If the areas of failure are few and not extended, it would be uneconomical to consider extensive repairs. In such cases it could be better to perform a local holding action to bring that specific areas on par with the rest of the pavement.
It should, however, always be considered that, where failure of limited areas start to manifest, the total pavement may be reaching the end of its service life. To now make expensive repairs on a pavement that is starting to show signs of overall failure can be a waste of funds. If funds allow it, total rehabilitation of that section of road may be the most economical option.

OPTION4: Total rehabilitation.

When it is found that considerable areas of the pavement is showing signs of distress during the structural evaluation, the total road section should be considered for rehabilitation involving replacing or re-working of the pavement layers. When such a section of road has been identified from the network analysis, further project level investigations and analysis need to be done to establish areas with similar problems (uniform sections). These procedures are described in the rest of this document.

Where the specific section of road is considered for total rehabilitation involving reconstruction of all or some of the pavement areas, one should not rush into such reconstruction unless the pavement is in such a state of disrepair that it creates a traffic safety risk. The required rehabilitation can sometimes be postponed until it really becomes a necessity to get as much of the remaining life out of the pavement as possible, seeing that it will be totally destroyed when rehabilitated.

3 PROJECT LEVEL ASSESSMENT

3.1 Introduction

A project, for our purpose, is defined as the pre-determined section of road that has been isolated out of the total road network that needs to be further investigated with the eye on more extensive pavement rehabilitation.

The data obtained during the network level assessment can be used for the project level assessment but certain tests, such as deflection tests, are usually done at too low frequencies to provide sufficient information on a project level. As riding quality, rut depth measurements and skid resistance tests are done at very short intervals during the network assessment, it would not be necessary to repeat them on a project level.
On the other hand, deflection tests are fairly expensive to do and on a network level is usually done every 200m. For proper analysis of the pavement to be done on project level, it is suggested that the test frequency be reduced to, say, 50m distances depending on the funds available for investigation and the longitudinal homogeneity of the pavement condition. In this section the various evaluation tests to be done will be discussed.

To illustrate all the steps in the process, a recently rehabilitated section of road on the N4 between Pretoria and the Mozambique border, namely N4 Section 3 from the Gauteng / Mpumalanga border towards Witbank has been chosen as an example. The section of road is 19,61 km long. Network level assessment has indicated that this section of road is in need of structural strengthening.

3.2 As-built pavement data.

To be able to evaluate the deflection results from the FWD tests, it is necessary to know the as-built materials properties and layer thicknesses. The pavement design data can be used but invariably would differ from the as-built data. So, for instance, may the design call for the selected subgrade to be 150mm of a G7 material. The borrow-pits used during construction could have been of considerably better, or worse, quality than asked for. Similarly, a density of say 93% of Mod AASHTO could have been specified but, due to the workability of the material used, typical densities of say 97% may have been obtained. In such cases it would be much easier for the designer to make adjustments to the layer properties in his pavement model derived from the deflection analysis.

If the as-built pavement data is not available, more tests pits will have to be opened in the road pavement to determine the materials quality of the layers as discussed later.

On the specific section under investigation, the construction and maintenance history is indicated in Figure 3.1. As can be seen, there is a marked difference between the pavements from km 0,0 to 14,4 and km 14,44 and 19,61.

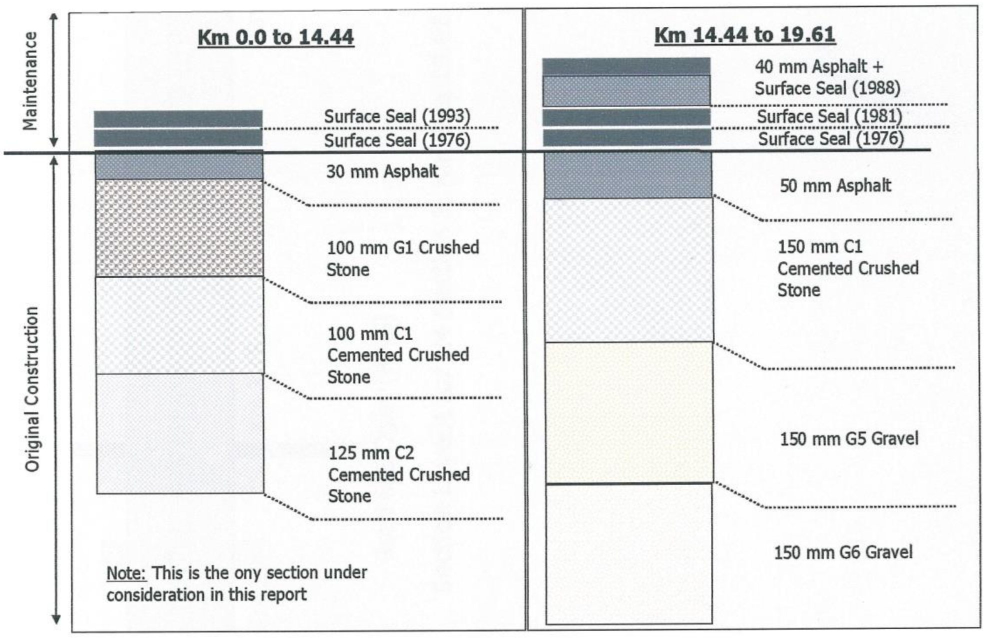


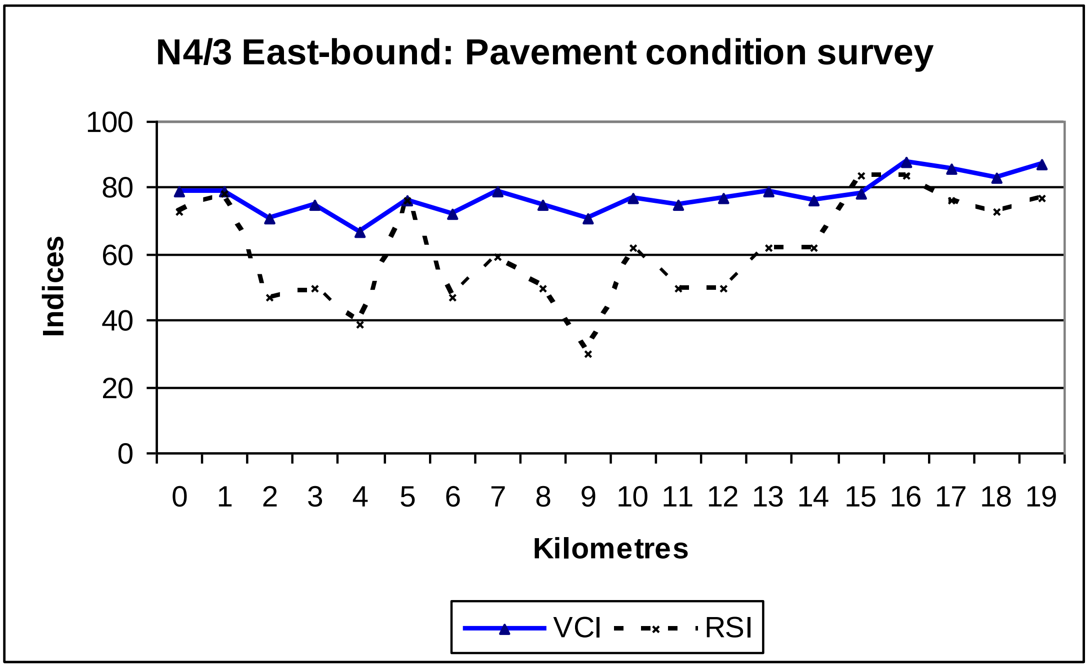
Figure 3.1: Pavement structure showing maintenance history.

3.3 Detailed visual inspection

The network visual inspections are usually done at a high speed and are not necessarily very accurate. To be able to make a proper evaluation of the road condition, it would be necessary to do a more detailed visual inspection using TRH9[2] as guideline. During the inspection aspects such as shoulder condition and shoulder drop must also be incorporated.
The road verge condition must also be reported on and edge breaks recorded for rehabilitation.

From the visual assessment of the section under investigation, the Visual Condition Index (VCI) and Re-seal Index (RSI) have been calculated using TRH 22 [3] algorithms for the calculations. Only the East-bound carriageway is shown as an example.

Figure 3.2: Visual condition index and re-seal index.



Each road authority would have their own standards for the VCI and re-seal index. For our purpose the following limiting value have been chosen:

Visual Condition Index: 60
Re-seal index: 70

On the graph, Figure 3.2, it is clear that the visual condition of the road is still acceptable but that nearly the total section between km 0 and 15 needs to be re-sealed or resurfaced.

3.4 Drainage conditions

Many sections of roads fail due to inadequate drainage of the subsoil. During the detailed visual inspection the drainage conditions must, therefore, be investigated. Inadequate subsurface drainage will invariably manifest itself in pavement failure, especially in cuttings. Experience has shown that the replacement or installation of subsurface drains will largely solve pavement problems in cuttings.
During the drainage inspection the drainage culverts and pipes should also be inspected to ensure that it is clean and working properly. Where metal culverts have been installed, these should be inspected for corrosion, etc. All in and outlets should also be inspected to determine if extensive cleaning thereof should be specified in the rehabilitation contract.
Similarly should side drains, down-shoots, etc be inspected to consider replacing or fixing them during the contract.
Erosion of fill and cut batters should also be evaluated and the reparation thereof included in the rehabilitation contract, if required.

3.5 Deflection testing

3.5.1 Background.

A convenient way of assessing the structural integrity of road pavements is to apply a load to the pavement surface and then to measure the resulting deflection of the pavement under the load, similar to what would happen below a loaded wheel travelling over the road.

Pavement deflection measurement techniques are numerous and can be categorised as follows:

Static or slow moving measurements:	Benkelman Beam, LaCroix Deflectograph (Fig 3.3).
Dynamic vibratory loads:	Dynaflect and Road rater.(Fig 3.4)
Dynamic impulse loads:	Falling weight Deflectometer. (Fig 3.5 and 3.6)
Moving vehicle:	Rolling wheel Deflectometer (FHWA) (Fig 3.7)

Figure 3.3: LaCroix deflectograph

Figure 3.4: Road rater testing a concrete pavement

Figure 3.5: Falling Weight Deflectometer

Fig.3.6 Geophone lay-out on the FWD

Figure 3.7: FHWA Rolling Wheel Deflectometer.

3.5.2 The Falling Weight Deflectometer (FWD)

Of the above-mentioned machines, the FWD is still the most frequently used deflection testing apparatus and especially in SA. The development of the next generation of deflection testing machines is underway with the FHWA Rolling Wheel Deflectometer (Fig 3.7). The machine measures the maximum deflection as well as the deflection bowl while moving at approximately 50km/h using laser beams between and in front of the moving wheel of a truck. The machine is busy being extensively tested in the USA and could soon be available in South Africa.

In the FWD, a circular mass is dropped from a certain height onto the road pavement imparting an impulse load of 40kN (a pressure of 560 kPa) onto the road surface. Using a set of geophones resting on the road surface at fixed distances, the deformation of the road is measured at those points (Figure 3.6).

Network level deflection tests using the FWD are usually performed every 200m of road length. This would, however, not provide enough detailed information to ensure that all possible areas to be isolated for rehabilitation would be adequately covered in the survey. For project level assessments it should be considered to do extra deflection tests every 100m, or even every 50m of road length. For single carriageway roads, the tests can be done every 100m in one direction and then staggered every 100m in the opposite direction. Deflection tests are usually only done in the slow lane left wheel path of the road. If the fast lane is suspect a few deflection tests can also be done there to cover all possible pavement problems. Usually the fast lane of a dual carriageway road or climbing lanes of single carriageway roads carry mostly light vehicles and, in most cases, seldom more than 20% of the traffic in that direction. The structural condition of the fast lane is, therefore, usually much better than that of the slow lane except when something unforeseen has happened.

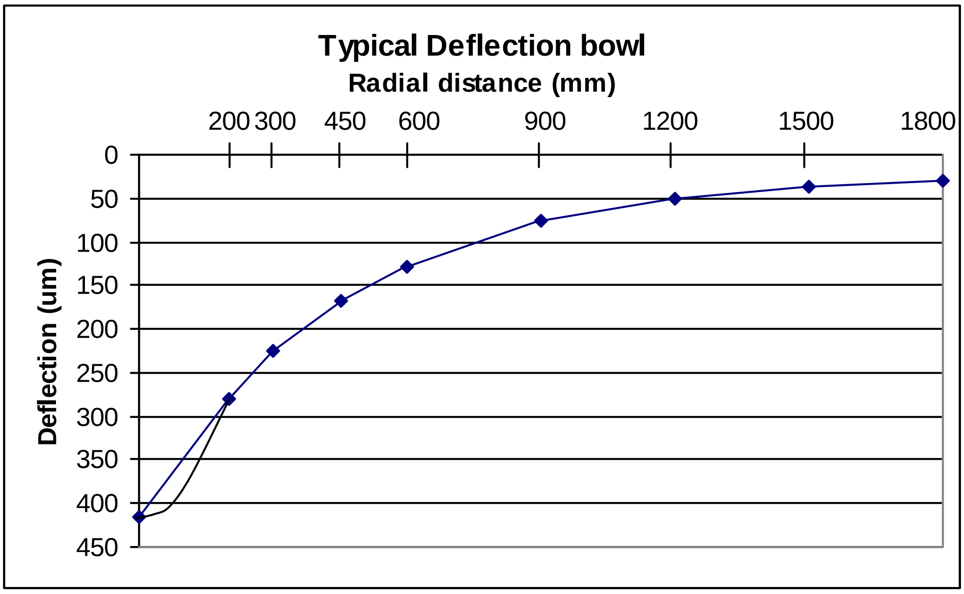
3.5.3 Evaluation of results.

The results from the deflection measurements can be used to serve as an indicator for further testing of the pavement to be carried out as well as provide sufficient data to decide on the rehabilitation options to be employed.

When a load is placed on the surface of the pavement it deflects downward to form a bowl shaped depression known as a deflection bowl or deflection basin. The size, depth and shape of the basin is a function of several variables, including the thickness and stiffness of the pavement, the underlying materials and the magnitude of the load.
The deflections measured by the radially spaced sensors defines the basin. Studies have shown that the outer deflection sensors respond primarily to the subgrade characteristics, while the inner sensors respond to the subgrade and upper pavement layers. These parameters thus allows one to estimate the stiffness profile of the pavement with respect to the depth below the surface.

In Figure 3.8 a typical deflection bowl taken from real data is shown. In practice the shape of the bowl at the zero distance point would be rounded as shown on the added drawn line when a tyre passes over the zero point and deflection is measured at that point.

Figure 3.8: Typical deflection bowl



However, the FWD foot plate, although segmented, is 300mm in diameter with a 50mm hole in the centre where the geophone sits. The rigid surroundings thus make it impossible to determine the shape of the bowl in the vicinity of the Zero point

Over the years numerous techniques have been developed to analyze deflection data from the different kinds of deflection equipment. In South Africa the most frequently used deflection device is the FWD and the focus, for our purpose, will thus be on analysing FWD data.

Various analysis techniques have been developed that can be used to evaluate the FWD deflection data. One such technique is the analysis of the deflection bowl or deflection basin. In table 3.1 the currently used deflection basin parameters are given that can be used to give an estimate of the structural integrity of the pavement.

Table 3.1 Most commonly used deflection basin parameters in the RSA

Basin Parameter	Calculation
Ymax = maximum deflection (μm) BLI = Base Layer Index (μm) MLI = Middle Layer Index (μm) RC = Radius of curvature	D_0 D_0-D_{300} D_{300}-D_{600} $\frac{(D_0-D_a)^2+r^2}{2(D_0-D_a)}$
D_x = FWD Deflection measured at an offset of x mm from the centre of the loading plate. D_a = FWD deflection at the edge of the loading plate. As there is no sensor to measure the deflection at the edge of the plate, the deflection is calculated as follows: $D_a = 0{,}125D_0+1{,}125D_{200}-0{,}25D_{300}$ (a=150mm for the FWD)	

Bound in these notes as **Appendix C** is a chapter from Research Report RR 93/296 (6) by Dr Rohde giving a more detailed description of the use of FWD deflections in the rehabilitation design of flexible pavements. The same approach is followed in the TRH12, Pavement rehabilitation manual. It is recommended that this report be studied and used for that purpose.

However, as an **initial** indication or departure point for further investigations, it has been found that the Maximum deflection values can serve as indicators of the structural integrity of a pavement. First of all, the normalised deflections are tabled in a spread sheet. As an visual indicator of the deflections, it is recommended that they be plotted. In Figure 3.9 such a plot is shown where deflections were done every 50m on project level.

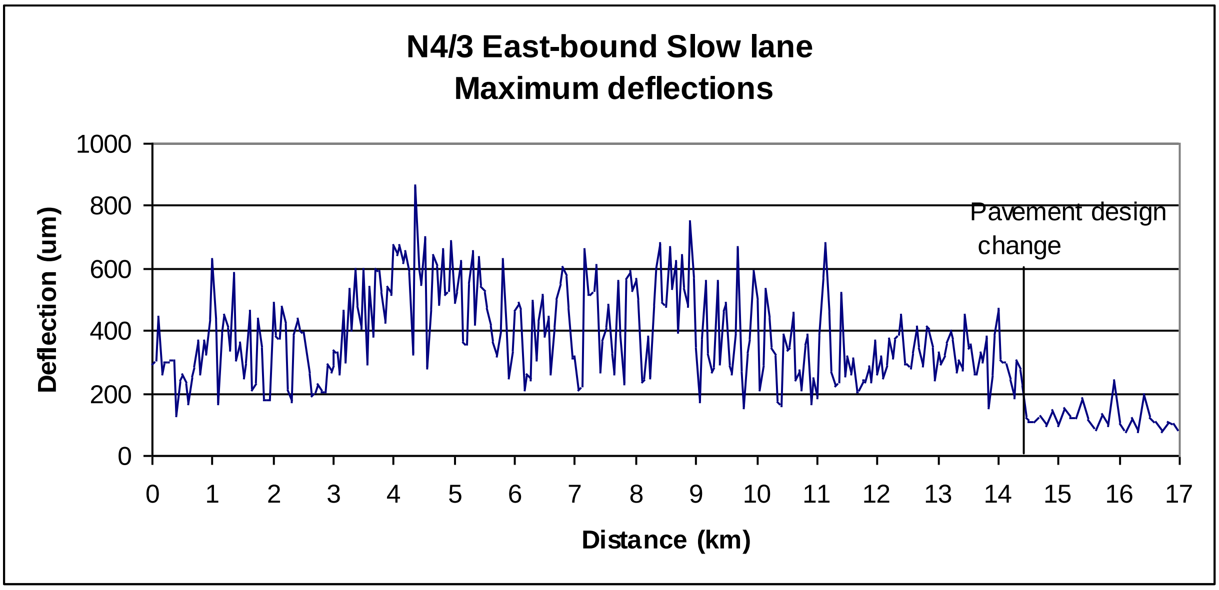


Figure 3.9: Plot of maximum FWD deflections

From this plot, it can be clearly seen that the section of road can be divided in different smaller sections with similar properties. It will be further expanded on when the division of the road in uniform sections is discussed.

3.6 Field investigation.

With field investigation is meant further on site and in-situ type of investigations and tests that can be performed to gather more information for further analysis and to be able to properly design a rehabilitation strategy.

3.6.1 Pavement design and as-built data.

As can be seen from the deflection plot of our example section, there is a mark difference between the deflections before and after km 14,5. The as-built data indicated that the design of the pavement changes at that point and that the next section of road was also constructed much later than the first section. (See figure 3.1)
Based solely on the deflection data, it is obvious that the pavement in the first 14,5 km of road needs structural strengthening in areas whereas the section after km 14,5 still appears to be in a good structural condition. This differences are obvious in Figure 3.1.

3.6.2 Test pits and DCP investigation.

The classic saying goes that one test is worth a thousand expert opinions. It is still valid today. Even with the most sophisticated non-destructive testing techniques, the designer would still require a better understanding of the nature and quality of the in-situ pavement materials.

Firstly it must be decided where these test pits have to be made. Testing is quite expensive and although one would want to make as many holes as possible, it is just not practical and economically justifiable

Although one can decide on a regular test hole pattern such as a hole every, say, three kilometres, the maximum deflection data plot in 3.5 can be used for that purpose.
In this case it is assumed that the pavement design is the same for the whole section. If the pavement design would vary, each such section must be regarded as a separate section.
As one wants to know the properties of the in-situ material in "bad" areas but also compare it with the material in a "good" area, it is suggested that test pits be made in both. Taking the deflection data plot again, it is suggested that test pits be made in the indicated areas for our example section.

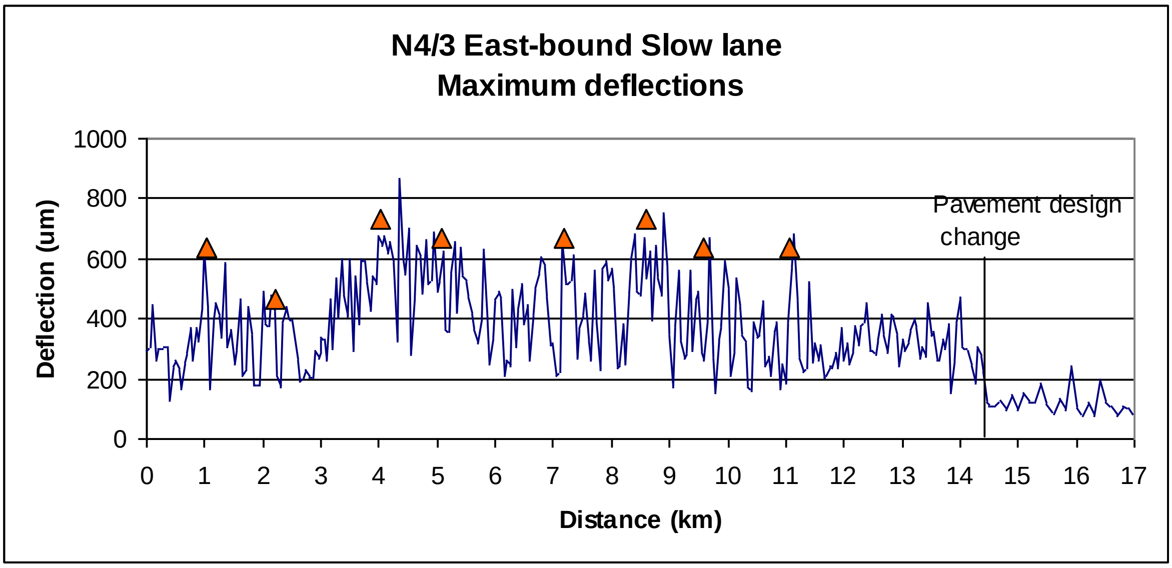


Figure 3.10: Indicating positions of test pits.

The test pit positions indicated in Figure 3.10 has been arbitrarily chosen in areas of high deflections and covers more or less the total road length. The hole at km 2,35 is situated in a area showing better properties and will be used as a control. Alternatively, the information obtained from the cusum plot of the deflections, discussed in 4.8, can be used for this purpose and a test pit made in each, so-called, uniform area.

Before the list of test positions is given through to the laboratory, it would be wise to check the test hole positions in the field to make sure that there would not be a problem if the holes are made there as some positions may be on a culvert or geometrically difficult position. If this is the case another nearby position with a similar deflection or in the same uniform area can be chosen.

Secondly, a **reputable laboratory** should be used to make the test pits and do the required tests.
They must arrange for suitable traffic accommodation measures to be taken with sufficient road signs in accordance with the SA Road Traffic Signs Manual, Chapter 13: Roadworks Signing.
They must also have enough suitable sample containers, cement and water for stabilising the back-fill material and cold asphalt for fixing the holes after testing and sampling.

Usually one would make a test pit approximately 1 x 1,5 m using hand tools or a small breaker if necessary. The hole is usually made next to the left-hand yellow line to go into the left wheel track. The main reason for this is to make traffic accommodation easier and to cover the area of pavement carrying the heaviest loads.
The hole must be square and sides straight. Firstly the surfacing is carefully removed and a sample taken for analysis, if deemed necessary. After the removal of the surfacing a **Dynamic Cone Penetrometer (DCP)** tests are done diagonally in opposite corners of the hole. If the DCP has difficulty penetrating one or more of the pavement layers, such as a stabilised subbase layer, that layer is first removed and DCP tests then done on the layers below.

The test pit is dug, layer by layer to be able to sample each of the existing pavement layers separately. The material is described as indicated below.

After removing the stabilised layers, the sides of these layers should also be tested for the presence of stabiliser by spraying it with diluted hydrochloric acid and phenolphthalein solution.
The layer thicknesses are measured as accurately as possible.
It is also recommended that photos are taken of each hole to see the layers and serve as a complete record of the survey.

The following information should be recorded for each hole;

Position: kilometre, Distance from edge of road, etc.
Surfacing: Type, thickness, cracked, fatigued, colour, dryness or brittleness.
Base; Type of material, colour, wetness, firmness, thickness.
Subbases: Type of material, cemented or not, colour, wetness, firmness (hardness), reaction with hydrochloric acid, colour with phenolphthalein, thickness.
Selected subgrade: Type of material, colour, wetness, firmness. (If stabilised same as above), thickness
Subgrade: Type of material, colour, wetness, firmness, thickness.

A typical photo of the test pit made at km 4,0, Eastbound on N4/3 (our example section), as chosen above is given in Figure 3.11

0 – 35mm: Asphalt plus two seals, intact, deformed. slightly, 3mm in wheel track.
35 – 170mm: Slightly moist, light brown, medium dense crushed granite, intact, imported.
170-266mm: Slightly moist light brown medium dense granite gravel, stabilised, intact, imported.
266-341mm: Slightly moist, brown, loose to medium dense felsite gravel, intact, imported.

341-502mm: Slightly moist, brown, loose to medium dense felsite gravel, intact, imported.

502-598mm Slightly moist, brown, loose to medium dense felsite gravel, intact, imported.

Figure 3.11: Photo of test pit km 4,0 EB

After sampling the pavement layers, measuring and photographing the test pit, it should be properly closed by compacting the lower layers at optimum moisture content up to approximately 400mm from the top. The top 360mm is then filled with the material removed from the test-pit plus extra material, usually crushed stone base material, required to properly fill it, mixed with approximately 4% cement, properly mixed and compacted in layers of not more than 100mm at optimum moisture content to as high a density as possible with hand compaction equipment. 1,5% of 60% emulsion can also be added to the material before mixing to aid in compaction and make the material less water susceptible. After compaction of the top layer, it is primed with diluted bitumen emulsion. The asphalted or sealed sides of the hole are painted with emulsion to approximately 50mm outside the edge of the hole. The asphalt or sealed surface is replaced with, preferably, hot asphalt, levelled and compacted to be flush with the surrounding road surface. Good quality cold asphalt can also be used but it should be monitored afterwards as it tends to deform under traffic.

3.6.3 Testing of samples.

The test-pit samples should be tested in a certified laboratory for at least the following:
Indicator tests (Grading analysis, Atterberg limits and Plasticity index, Grading modulus)
Strength tests: CBR on similar materials.

Usually not all the material in all the test pits is tested for CBR to limit expenditure although enough samples are taken to do so if required. After the grading and Atterberg results are known, materials that are obviously similar are mixed together and tested for CBR.
This obviously involves a certain amount of risk but the onus rests on the designer to decide on the number of tests to be done. These tests, in any case, only gives an idea of the material involved to aid in calibrating the back calculation model.

3.6.4 DCP Analysis

The DCP tests (Dynamic Cone Penetrometer tests) results are also analysed by plotting the penetration against depth and calculating the DCP number or in-situ CBR number or whatever is to be used by the designer.

An example of such a DCP analysis is presented in Appendix B.

The DCP software that was used to develop the curve is the Rubicon Toolbox pavement analysis software from Modelling and Analysis Systems [8] but other similar software packages are available. The curve can also be plotted using a normal spreadsheet.

In the DCP analysis the estimated stiffness values are given that can also be used, in conjunction with the results from the laboratory tests, as limiting moduli for the back-calculation of the FWD deflection results.

In our example pavement, the DCP failed to penetrate the base and subbase layers. These layers were, therefore, removed before the DCP tests were done on the underlying layers.

4 PAVEMENT ANALYSIS

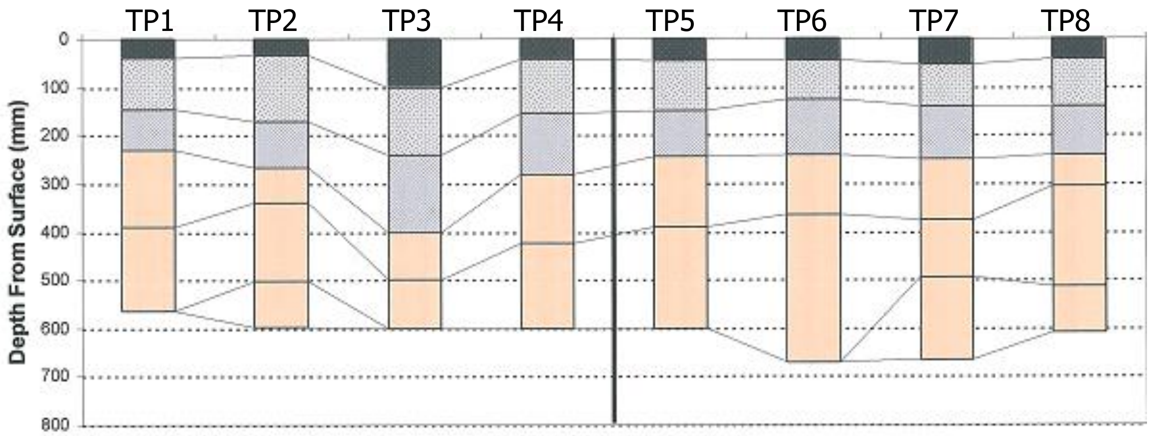
4.1 Compilation of data

Before the pavement analysis can be done, it would be necessary to compile all the field data collected during the investigations described in the previous chapter to enable a logical rehabilitation strategy to be decided upon.

The following important information is required:

Layer thicknesses:
In the back-calculation procedure to determine the modulus or stiffness of the individual layers, the accurate thickness of the layer is of prime importance.
The original surveyed as-built layer thicknesses are virtually never contained in the as-built data with the result that it has to be obtained from the test pits dug in the road.
To take our analysis section as an example, the test pit layer thicknesses are graphically presented in Figure 4.1.

Figure 4.1: Layer depths obtained from test pits.



It is obvious that test pit no.3 is an outlier. It seems as if an overlay of some sorts was placed in that area which resulted in a 100mm asphalt layer.

From the above the layer thicknesses can be obtained.

Materials classification:
From the test results of the different layers, the materials must be classified in terms of TRH14 (or TRH4) as indicated in **Appendix A.**
As only a few test-pits are usually made in a rehabilitation section and the test results may vary widely, classifying the material accurately for any specific layer in the pavement, in a specific class, is not always easy. A methodology currently being developed uses the fuzzy logic theory for more a consistent classification of the material. It is based on the certainty of the accuracy and representation of a specific test and then using that certainty to class the different results. The method was initially developed for classifying material for bitumen stabilisation but is just as applicable for any rehabilitation work. Details of the procedure can be found in a Technical memorandum by F Jooste, F Long and A Hefer called: *"A Method for Consistent Classification of Materials for Pavement Rehabilitation Design"*. The procedure is also described in the TG2 manual: *"A Guidelines for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials"* published by the Asphalt Academy. The

first documents can be downloaded from the Gautrans website with reference: "CSIR/BE/IE/ER/2007/0005/B" If one "Googles" this reference it will be found on more than one site.
The TG2 manual can be downloaded from the Asphalt Academy website.
An software program to do the material classification is available on www.aspahltacademy.co.za/bitstab. The data is prepared in an Excel spreadsheet, using a template available on the website, and then run on the website. The principles of the procedure is explained in **Appendix A.**

As mentioned previously, this classification is accurate enough to be used to identify and categorize materials.

Determination of stiffness moduli limits.
It is necessary, when using any of the back calculation computer programmes or even when doing it by hand, that the modulus values calculated for each of the layers are logical and complies with the type or class of material present in that layer. For that reason it is necessary to give lower and upper stiffness value limits within which the computer can look for applicable stiffness values to estimate the deflections.
Formulas can be applied to estimate elastic stiffness (elastic modulus) values from the CBR and other tests. Research has, however, indicated that by categorizing the material in terms of the TRH 14 (or TRH4) classes, stiffness values can be derived that are accurate enough to calibrate the back-calculation of modulus values from deflection data or to determine the stiffness modulus limits in the back calculation procedure. [6][7]

With reference to the TRH14/TRH4 materials classification given in Appendixes A-1 and A-2, the Stiffness (Modulus) values in Table 4.1 can be used for the different classes of materials: [6][7]

Table 4.1 Estimated Modulus limits to be used for pavement materials

Materials Code	Material	Estimated Modulus value
NATURAL	MATERIALS	
G1	Graded crushed stone	300-450
G2	Graded crushed stone	250-400
G3	Graded crushed stone with soil binder	230-350
G4	Crushed or natural gravel	150-300
G5	Natural gravel	130-250
G6	Natural gravel	120-150
G7	Gravel/soil	60-90
G8	Gravel / soil	40-80
G9	Gravel /soil	30-60
G10	Gravel /soil	20-50
TREATED	MATERIALS	
C1	Cemented crushed stone or gravel	7500
C2	Cemented crushed stone or gravel	5500
C3	Cemented natural gravel	2500
C4	Cemented gravel	1500
AG	Gap-graded asphalt	2500
AC	Continuously graded asphalt	3000
AS	Semi-gap graded asphalt	2500
AO	Open graded asphalt	1500
AP	Porous or drainage asphalt	1500

Test pit data.
All other test-pit data should be compiled in such a manner that it can be represented in a report. Some of the software programs mentioned below can be used to assist with that.

DCP data.
The data obtained from the DCP analysis shall also be presented in the report for use during comparative stiffness analysis and are also integrated in the knowledge based system described in Appendix A.

Deflection data
If a software program such as Rubicon Toolbox® (8) or PADS 2007 (10) is used to do the back-calculation of the deflection data, the raw FWD data should be placed in a standard spreadsheet in the format required by the program.

4.2 Isolation of key design issues.

In order to aid with the final decision on a rehabilitation strategy and to determine uniform sections for rehabilitation, key issues emerging from the visual survey, test-pit observations, etc, must be summarized. This information can also be used to determine the limits of modulus values for the back-calculation procedure.

In our example, the key elements of the test-pit observations are:

- A thin 35-52mm asphalt layer, plus two surface seals was noted in all holes except Test pit 3
- The base and subbase show as a light brown to light grey zone in the structure. These layers generally consist of crushed granite.
- The selected layers are indicated by a distinct reddish brown zone in the pavement structure.
- The asphalt surface has rutted and is showing fatigue cracked due to ageing.

- Grading and other indicator tests performed on the materials indicated the following:

  - Gradings of the base material conform to the grading envelopes of G1 to G3 material with a small percentage of oversize material if compared with the G1 grading requirements specified in the COLTO standard specifications.(9).

  - The cemented subbase material generally matches the grading requirements for G4 material but not for crushed stone material based on the TRH14 recommendations. Although the subbase was designed as a C1 and C2 material, it appears to fall in the C3 to C4 category, except for Test Pit 2 where a much lower deflection value was observed.

  - The presence of cementitious stabiliser could only be proven in two of the eight test holes. If one considers that the test pits were made in areas where the pavement condition, based on maximum deflections, were the worst, this finding could indicate why the subbase failed.

  - The DCP results indicate that the selected layer(s) are sub-standard to good with in-situ CBR values of between 30 and 90. The upper subgrade layer is in a good condition with a CBR in the region of 30%

4.3 Back-calculation at selected positions.

To be able to analyse the structural capacity of the pavement, a deflection bowl analysis on all data can now be done. But, to calibrate the model using the test-pit and DCP data, it is better to manually back-calculate the deflection data closest to the test pit positions.

Either of the back-calculation software programs mentioned in 4.1 can be used for this purpose. Before using any of these programs, the user should familiarise himself with the workings of the program, the benefits and shortcomings thereof and input format. The procedures described in **Appendix C** can also be followed.

As most of the back calculation programs can only handle five layers, it is sometimes necessary to combine some of the layers to limit the total to five. It is important that the limiting modulus values of each such layer or layer combination is estimated from the values suggested in Table 4.1 to make the analysis easier, quicker and more accurate.

In figure 4.2 the back calculation results from our example is given. In the example, the following was decided for the **initial** analysis:

- Because of the low thickness of the asphalt layer, the asphalt stiffness was not varied in the analysis but fixed at 2500 MPa.
- In view of the relative similar nature and low individual thicknesses of the crushed stone base and deteriorated cemented subbase, the two layers were combined and modelled as a single layer.
- All selected layers were modelled as a single layer.
- The subgrade was modelled as two layers with the top layers as a 1200mm thick layer and the bottom layer as semi-infinite.

Rubicon toolbox has been used to analyse the data but PADS or any similar program can also be used.

If it is found during the back calculation of individual points that stiffness values outside the expected stiffness range for that material is obtained, the layer combinations can be adjusted until a satisfactory result is obtained.

Deflection Bowl Report At Station 4.05

Layer Thickness = 40 Millimetres
Layer Stiffness = 1688 MPa
Poisson's Ratio = 0.4

Layer Thickness = 208 Millimetres
Layer Stiffness = 323 MPa
Poisson's Ratio = 0.35

Layer Thickness = 365 Millimetres
Layer Stiffness = 70 MPa
Poisson's Ratio = 0.35

Upper Subgrade:
Layer Thickness = 1200 Millimetres
Layer Stiffness = 162 MPa
Poisson's Ratio = 0.35

Lower Subgrade or Substratum is Semi-Infinite, with Stiffness = 790 MPa

Deflections:

Offsets (Millimetres):	0	200	300	450	600	900	1200	1500	1800
Measured (Microns):	642	487	325	177	99	43	28	19	14
Calculated (Microns):	640	380	267	169	109	46.1	21.6	11.9	7.89
Error at Sensor (%)	0	20.9	16.2	0	0	0	0	0	0

Average Error Per Sensor: 4.13 % **Sum of Errors: 37.2 %**
Note: Errors were adjusted for precision. Errors less than 10 micron are shown as zero.

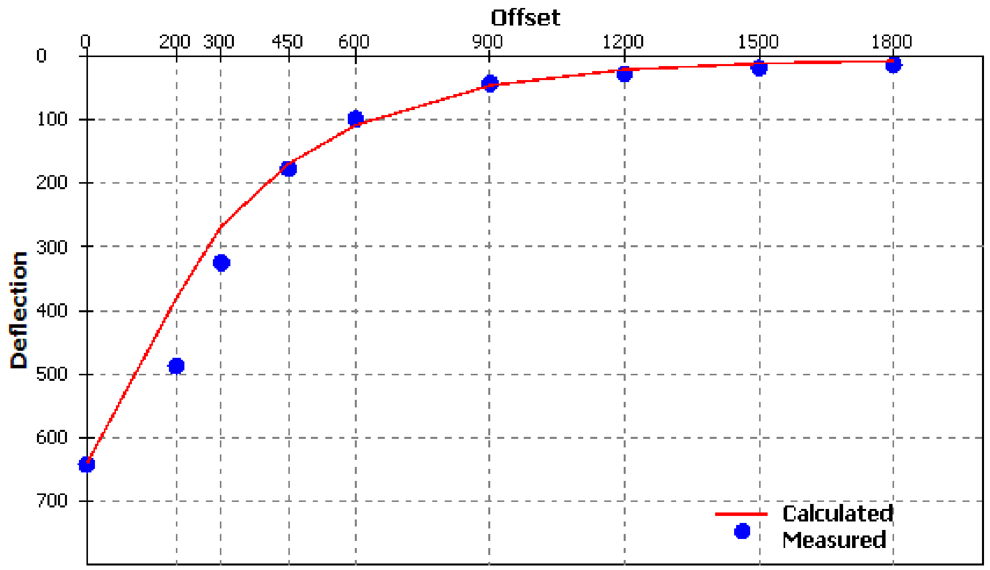


Figure 4.2: Measured and calculated deflection bowls

Rubicon Toolbox: Deflection Bowl Analysis / Ver: Undefined / (Unlicenced)

4.4 Back calculation of all data.

Automated back calculation of all FWD data can now be performed with the assumptions decided upon after the individual points were analysed.

- First of all the modulus search ranges for the individual layers have to be determined or decided upon.
- Secondly, it must be decided what layer combinations can be made to stay within the maximum of five layers to be analysed.
- Thirdly, the layer thickness of these layers or combinations must be decided upon.

For our example, the values in Table 4.2 have been used:

Table 4.2: Search ranges and layer thicknesses to be used for back calculations.

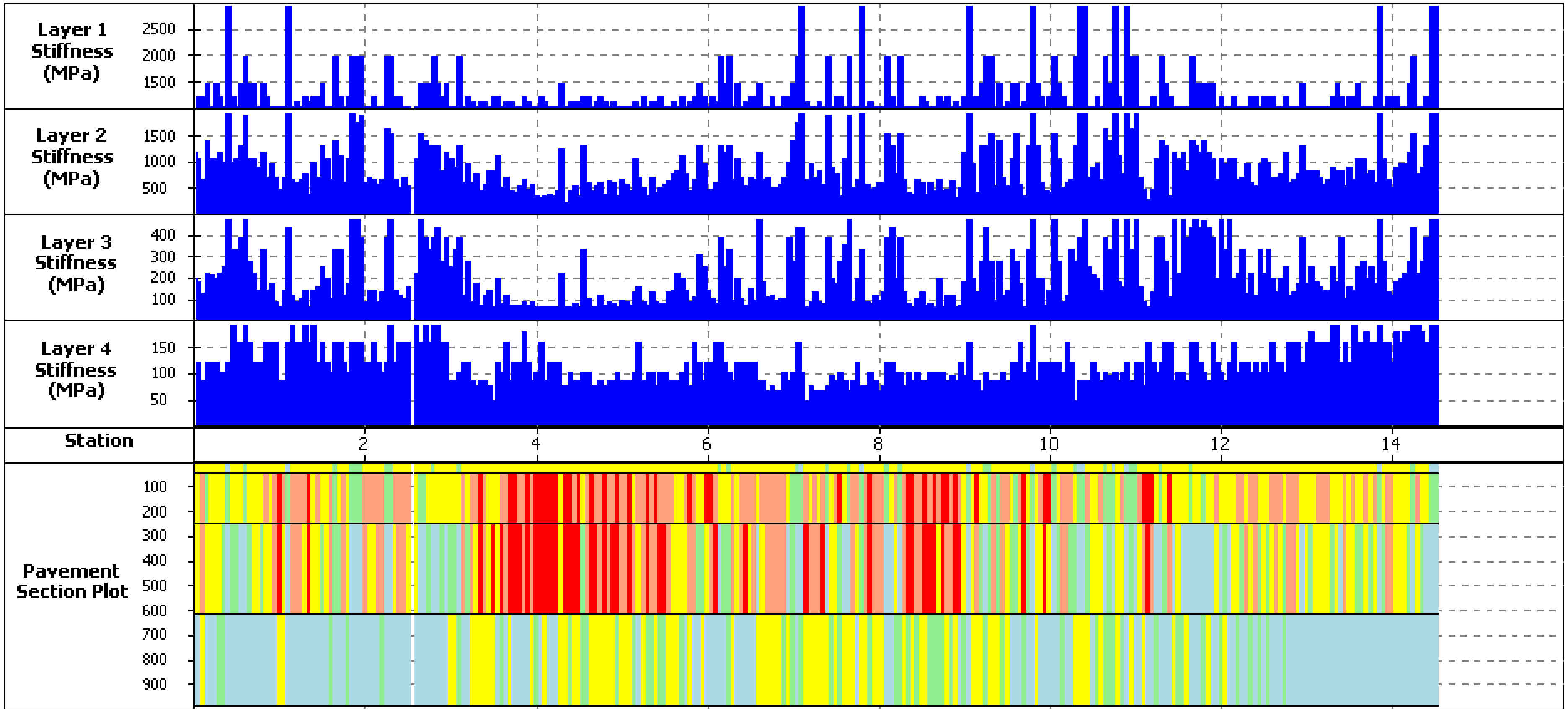
Layer	Layer thickness (mm)	Search range (MPa)		Poisson's ratio
		From	To	
Asphalt surfacing	40	1500	3000	0,4
Combined base and subbase	208	150	2000	0,35
Selected layers	365	70	500	0,35
Upper subgrade	1200	50	200	0,35

A back calculation was performed on our example for N4/3 km 0-14,5 using the Rubicon Toolbox and the results graphically expressed in Figure 4.3.

A decision must now be made on what is considered "good" or "poor" in terms of layer stiffness values. For this specific example, the ranges of values that were considered to be applicable are given in Table 4.3:

Table 4.3 Ranges used to interpret layer stiffnesses

Material	Interpretation ranges used (MPa)				
	Very good	Good	Fair	Poor	Very poor
Asphalt	3000-5000	1800-3000	1200-1800	800-1200	0-800
Crushed stone	500-2000	250-500	160-250	120-160	0-120
Selected layer	180-500	140-180	90-140	70-90	0-70
RSA Subgrade	150-500	90-150	60-90	50-60	0-50



Layer Stiffness Statistics (all values are in MPa)	Layer 1 Stiffness		Layer 2 Stiffness		Layer 3 Stiffness		Layer 4 Stiffness	
	Median = 1125	2th Perc. = 1000	Median = 786	2th Perc. = 367	Median = 178	2th Perc. = 70	Median = 125	2th Perc. = 68.8
	Std.Dev = 468	5th Perc. = 1000	Std.Dev = 429	5th Perc. = 410	Std.Dev = 134	5th Perc. = 70	Std.Dev = 36.6	5th Perc. = 78.1
	COV (%) = 35.7	10th Perc. = 1000	COV (%) = 46.6	10th Perc. = 468	COV (%) = 60.6	10th Perc. = 76.7	COV (%) = 29.8	10th Perc. = 86.6
	Minimum = 1000	15th Perc. = 1000	Minimum = 237	15th Perc. = 526	Minimum = 70	15th Perc. = 88	Minimum = 50	15th Perc. = 87.5
	Maximum = 3000	20th Perc. = 1000	Maximum = 2000	20th Perc. = 555	Maximum = 500	20th Perc. = 96.9	Maximum = 200	20th Perc. = 87.5

Figure 4.3 Back-calculation results N4/3 km 0-14,5

Rubicon Toolbox: Backcalculation / Ver: 2.6.3 / (Licenced)

4.5 Analysis of back-calculation data.

After the back-calculations have been performed and preferably plotted out as in the example, various deductions can be made from the results. This can be best illustrated with reference to our example.

From the back calculation results, various aspects can be observed namely:

- Due to the thin asphalt and seal layer, the results are not reliable. The asphalt surfacing has a median stiffness of 1688 MPa which falls in the "fair" category. A considerable percentage of these values, however fall in the "poor" category.

- When the entire section is considered, the stiffness of the combined base and subbase is generally above 500MPa which is considered to be of high stiffness if it is considered a crushed stone layer. If, however it is considered as a cemented combined subbase/base layer, the stiffness values are considered to be medium to low and typical for deteriorated cemented layers. In areas with higher deflections, the base/subbase indicated a markedly decreased stiffness

- The low stiffness of the base in areas where the deflections are high would be a contributing factor to the disintegration of the asphalt layer. It may also be that the poor asphalt in areas had a higher permeability causing water ingress into the base, resulting in a lower stiffness.

- The combined base and subbase layers have a widely varying stiffness. This may be caused by the cemented subbase which is still stiff and relatively intact in some areas but broken into smaller blocks (granular stage) in other areas.

- The selected layers generally show stiffness values of above 190MPa which is considered as of good quality. It also agrees with the observed DCP penetration rates for that layer. In some areas, however, such as between km 3,0 and 5,5 these layers have a significantly lower stiffness which may require special rehabilitation design considerations.

- The upper subgrade stiffness is generally above 100MPa with the 15th percentile value always above 80MPa, suggesting a fairly good subgrade. This observation is also in line with the DCP penetration rates.

If the back calculation results are further compared to the maximum deflection, it is indicated that the higher deflections are caused by a combination of lower subgrade stiffness and significantly reduced stiffness of the base and subbase as well as the selected layers.
An example of this is given in Table 4.4.

Table 4.4: Layer stiffnesses at low and high deflection locations.

Parameter	High deflection		Low deflection	
Station	3.35	8.4	12.15	1.5
Max deflection (micron)	775	808	222	251
Base and subbase stiffness (MPa)	266	258	1191	959
Selected layer stiffness (MPa)	70	70	392	231
Subgrade stiffness (MPa)	69	106	162	88

The pattern noticed in table 4.4 suggest that the dominant deterioration mechanism is weakening of the cemented subbase, with accompanying decrease in the effective stiffness of the crushed stone base and selected layers. Accelerated deterioration of the cemented layer may be caused by a weaker subgrade in some areas.

One should consider that all pavement layers interact in the pavement structure and that it is not always easy to decide which layer deteriorated first to cause the other layers to deteriorate. For example, lower subgrade stiffness may be partly caused by a weaker cover. Conversely, the weakening of the cover layers (particularly the cemented subbase) may be caused by a poor subgrade.

Considering the rutting results, the absence of considerable rutting suggest that the subgrade layers are well compacted and should form a good platform for the top layers. The indication is thus that the subbase layers deteriorated faster.

Also refer to Par 8.4.1 in **Appendix C**.

Also refer to the paper in **Appendix D**: Assessing Material Properties for Pavement Rehabilitation Design by Van Wijk, Harvey and Hartman.

4.6 Calculate the Structural Capacity.

To calculate the structural capacity or remaining structural life of a pavement, the stiffness (modulus) values of the different layers are used together with so-called transfer functions to estimate the remaining structural life of the pavement. This is best described in Par 8.4.2 and onwards in Appendix C. It can also be done by using a computer program such as Rubicon Toolbox® that includes all the required transfer functions currently being used and which is regularly updated from the newest research.

When the structural capacity of the road at the various FWD test points have been calculated, the next step will be to select areas of the road for rehabilitation by selecting so-called uniform sections.

As the output spreadsheet from Rubicon Toolbox is rather large and difficult to show, it has not been included in this presentation for our example section. However, from the chart in Figure 4.3, it is evident that certain sections are worse than the other i.e. the red coloured areas on the chart.

4.7 Alternative analysis.

4.7.1 TRH12 Method.

Software programs such as Rubicon and PADS are not always available to the designer. Alternative ways of analysing the data must then be used. One such procedure is to use the deflection parameters described in Table 3.1 and also presented in TRH12 as an indication of pavement structural strength. It is basically the same as explained in Appendix C.

To illustrate the use of these parameters, we will use our example section. The Maximum deflections, Base Layer Index (BLI), Middle Layer Index (MLI) and Lower Layer Index (LLI) have been calculated using the formulas in Table 3.1. It can be graphically illustrated in Figure 4.4. An alternative way of looking at these parameters is illustrated in Figure 4.5. In this figure, each of the above parameters is expressed as a percentage of the maximum deflection. (After Joe Grobler, VKE)

From these graphs the influence of each of the deflection parameters has on the total deflection can be quantified and is it possible to isolate the specific problem areas in a pavement contributing to the deflection.

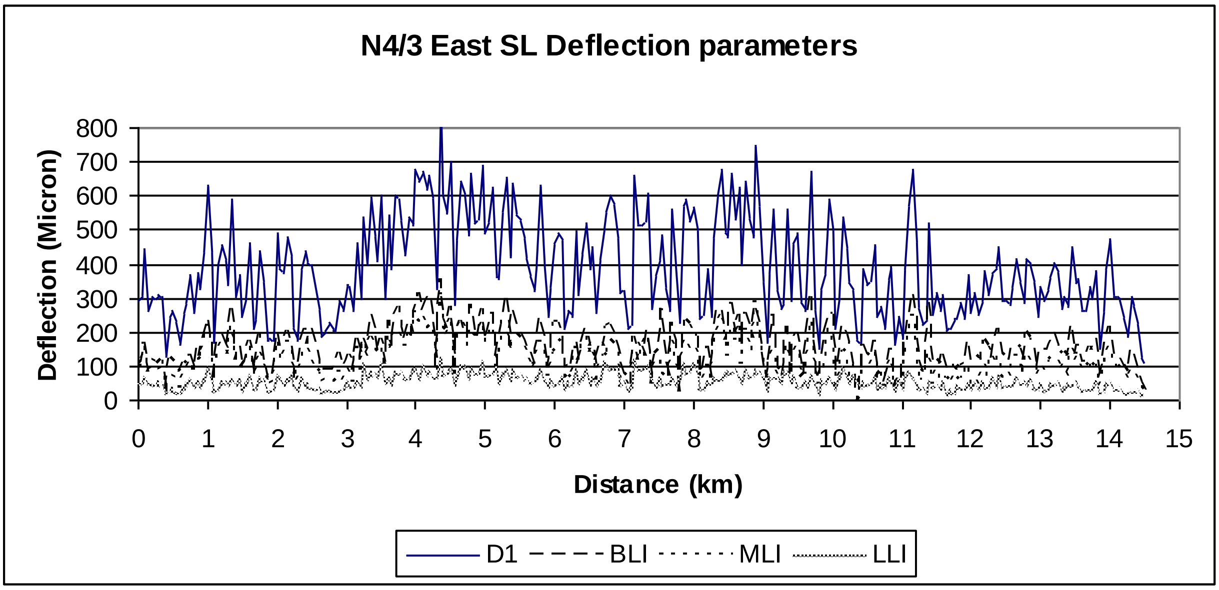


Figure 4.4 Deflection basin parameters plotted for N4/3 East-bound slow lane.

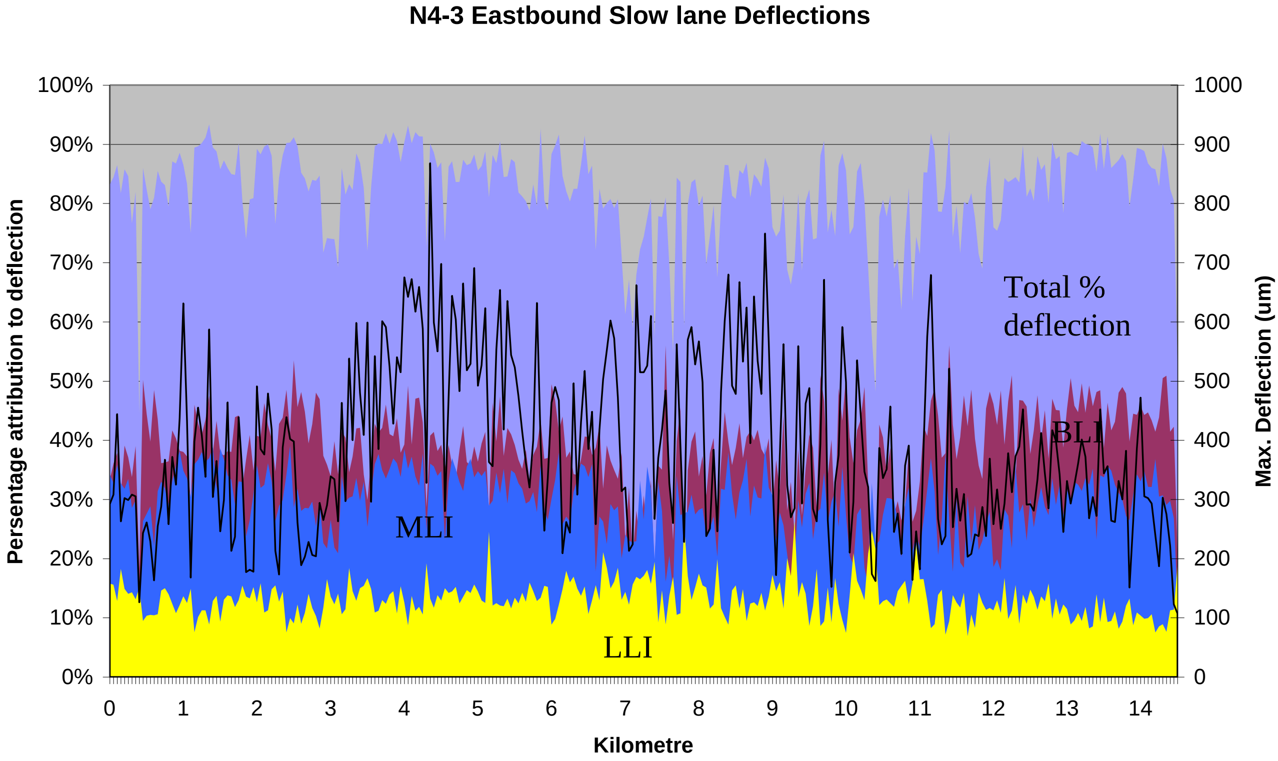


Figure 4.5 Deflection parameters plotted as a % of maximum deflection

In figure 4.5 it is obvious that the Middle Layer Index is higher for most of the section than the other indices, indicating that failure would be in the subbase area. Areas with the highest deflection can also be isolated for rehabilitation purposes.
One of the shortcomings of using the basin parameters is that one does not know the extent of the zone of influence of each so-called index. Layer thicknesses are not brought into the analysis and the differentiation between individual "layer" depth are thus vague. Comparing the results of the deflection parameters to the results obtained from the linear elastic analysis, some broad comparisons or general deductions can be made but not specific enough for a proper engineering decision to be made. So, for instance, is it impossible to deduce that the asphalt layer is failing from the deflection indices although it is clearly indicated in the linear elastic analysis.

4.7.2 The PN (Pavement Number) Structural Design Method.

This is a fairly new procedure developed specifically for the TG2, Bitumen Stabilised Materials Manual but is just as applicable to any pavement design. It is an amendment of the old AASHTO design procedure and has been locally developed and tested. The advantages of this method, as described in Appendix C of the TG2 manual are:

- **Data from in-service pavements** were used to develop the method. The type and detail of the data suggests the use of a relatively simple method and precludes the use of a Mechanistic-Empirical design method.
- The method gives a **good fit** to the available field data.
- The method is **robust**, and cannot easily be manipulated to produce inappropriate designs

The development and validation of the PN method are described in Jooste, *et al* (2007) and Long (2009). (The references refer to those in Appendix C of TG2.) The method is applicable to all pavement materials commonly used in southern Africa. This method relies on basic points of departure, or rules-of-thumb, which reflect well-established principles of pavement behaviour and performance, and which will ensure an appropriate pavement design solution in most situations. The concepts in the rules-of-thumb are quantified into specific rules with constants or functions associated with each rule. The rules-of-thumb are described in detail in Appendix C of the TG2 manual and will not be further discussed here. The method is available on the website of the asphalt academy:
www.asphaltacademy.co.za/bitstab
The PN number can also be determined, interactively, on the website. It also includes the calculation of pavement capacity. As the procedure is still, to a certain degree, under development, it is recommended that it be used in conjunction with the back-calculation procedure as a control.

4.8 Determination of uniform sections.

As mentioned before, it is necessary to divide the total road section under consideration for rehabilitation in uniform sections where the rehabilitation needs would be more or less similar.
In most instances this is done on a rather arbitrary way. The most practical method for the determination of uniform sections is by using the CUSUM procedure. The CUSUM plot of our example section is given in Figure 4.6.

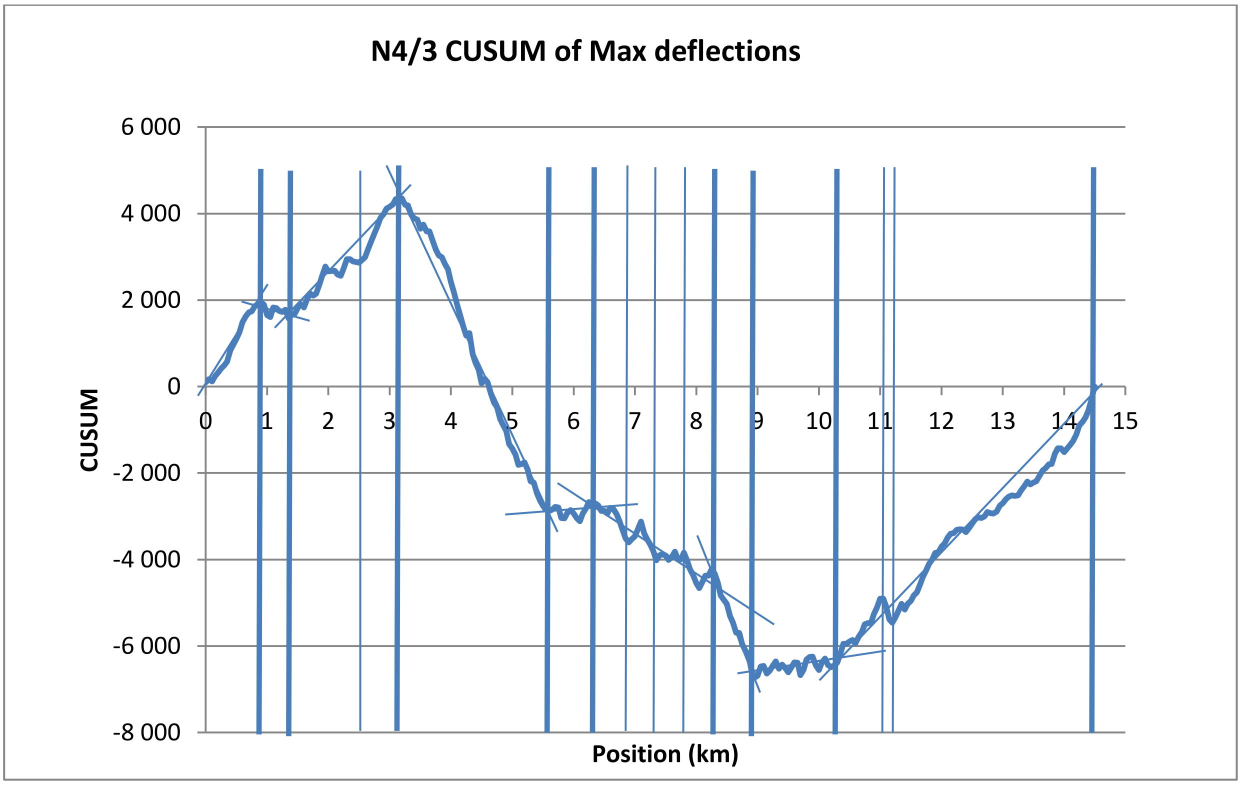


Figure 4.6: CUSUM plot of maximum deflection values.

The Cusum plot is compiled as follows, using a spreadsheet for convenience:
Determine the average of all the values to be plotted – in this case the maximum deflection values.
Subtract each of the values from the average.
Cumulatively add the values together in a separate column.
Plot the cumulative value at that point against the position on the road.

In the Cusum plot, a change of slope in the curve indicates a gradual change in the property being investigated. In Figure 4.6 the change in general slope is indicated by the vertical lines. One can, obviously mark any small change but the change should be realistic as was done with the wider lines. Even that may give too many sections to practically dealt with and should be further defined by looking at the general trend of other indicators such as rutting, etc. The back calculation stiffness values of the different layers can also be used as can be seen in Figure 4.3.

The following principles should be considered when uniform sections are determined:

Keep it simple and practical. Too many different rehabilitation options will make each action smaller and the rehabilitation contract more expensive. Some contractors specialise in only certain types of construction. If, for instance deep milling with stabilisation is considered, an earth-works contactor will most probably tender. If it also involves seal-work, he would most probably sub-contract a seal contractor. If asphalt is also included, a third sub-contractor would sometimes be required, and so on.
Small quantities of asphalt in areas where an asphalt plant is not close by can become very costly. Another type of construction should then be considered. The same actually applies to any type of construction where quantities are becoming small.
When doing deeper rehabilitation such as deep milling, replacing the base with new material or recycling, very short sections identified as still in a good condition between longer

rehabilitated sections should be eliminated. Rather spend a little more money and link these areas together even if a short good section is included in the rehabilitation action.
The width of sections to be reworked or replaced should be such that a recycler or other construction plant can easily work inside the area ensuring that the joint between the new and old pavement falls outside a wheel path.

5 SELECTION OF REHABILITATION OPTIONS.

5.1 Introduction

After the selection of uniform sections or specific areas in the road that would be in need of specific rehabilitation actions, the different rehabilitation options must be considered.

The selection of the specific rehabilitation option would largely depend on the funds available for rehabilitation.
Depending on the outcome of the analysis, the designer must decide on one of the following options:

- Do nothing now.
- Short term maintenance or holding action.
- Medium term rehabilitation option.
- Long term rehabilitation option.

If one considers that the normal design life of a new flexible road pavement is between 20 and 25 years, the following time scales can be attributed to the rehabilitation design options:

Short term maintenance or holding action:	2 to 5 years
Medium term rehabilitation option:	10 to 15 years
Long term rehabilitation option:	15 + years.

These are obviously rough guidelines and the road authority must decide on what maintenance strategy would best fit his budget and road network requirements.

5.2 Do nothing now option.

As mentioned in 2.3, the do nothing option will apply when all the functional and structural indicators comply with the functional and structural requirements.
If, however, it was indicated during the investigation that rehabilitation of a section of road is required but that the necessary funds are not available or the section is so small that rehabilitation at this point in time would be impractical, one option that can be considered is to do nothing for the time being. One should, however, consider the risk if this option is chosen. If, for instance, the areas affected are rather small compared to the total road area, the risk should be fairly small to leave the affected area for later rehabilitation or to consider the following option where a temporary holding action can be performed on that section.

Circumstances under which the do nothing option can be considered are the following:

- The road agency is prepared to accept a lower standard for the functional property that failed during the investigation. If, for instance, the maximum allowable rut depth on a section of road is 10mm and during the investigation it is found that the rutting is borderline or the average is just over the limit, the road authority may decide to temporarily accept a lower standard and postpone corrective actions to a later date.

- The section that failed is too small to warrant rehabilitation at this stage. In such a case a temporary holding action can be performed on the failed section to keep it functional until more extensive rehabilitation is required.

- If the structural evaluation indicates that a section of road is in need of structural rehabilitation but the visual inspection does not indicate visual failure, the rehabilitation can be postponed until the road visually fails or routine maintenance on that section becomes too expensive. One must remember that the parameters for structural failure only apply for standard cases. It is often found that a specific section of road show these standards not to be applicable. Spending money prematurely on such an outlier section must be carefully considered. One should, however, budget for the rehabilitation of the section as failure may be sudden and dramatic, calling for immediate action.

It should always be remembered that "a stitch in time may save nine" and that any larger scale rehabilitation can not be indefinitely postponed until the road is completely destroyed leading to extensive and costly rehabilitation.

5.3 Short term holding actions.

5.3.1 Basic principles

When funds are limited or for other practical considerations such as too short failed sections that do not warrant full scale rehabilitation, so called holding actions can be performed. It has been found that these so-called holding actions can extend the service life of a pavement with many years until larger or full scale rehabilitation becomes necessary. This type of treatment may also be used as a **preventative maintenance** measure. Obviously the risk associated with this type of treatment should be considered. It would be uneconomical to decide on a short term rehabilitation action when more than 50% of the pavement is in such a condition that full scale rehabilitation is indicated.

In Table 5.1 an indication is given of the extended service life that can be obtained from preventative or short term maintenance treatments (From the Federal Highway Administration). These treatments exclude the correction of some structural pavement problems such as excessive rutting or pavement failures, discussed further on and implies that structurally the pavement is still in an acceptable condition.

Table 5.1: Extended service life gains for preventative maintenance treatments of Flexible pavements.

Treatment type	Extended Service Life (years)
Bituminous enrichment spray.	1-2
Reinforced crack filling (For active cracks >6mm)	Up to 2
Crack sealing (for cracks 3-6mm)	Up to 3
Single seal	4 to 7
Double seal	5 to 8
Slurry seal	1 to 3
Ultra thin friction course	5 to 8
Asphalt overlay (40mm)	5 to 10
Asphalt mill and overlay (40mm)	6-12

The short term holding action would only include limited structural repairs of the pavement and usually entails the following:

- Mark out all sections that show signs of failure such as bad cracking, excessive rutting, deformation, potholes, etc.
- Do a DCP investigation or visually decide to what extend the failed pavement layers, if any, can be reconstructed with the available funds.
- Usually, in this type of rehabilitation, deep stabilisation would be ruled out as being too costly except in limited areas where doing anything else would be a waste. In such a case, the problem can most often be temporarily solved by removing and reconstructing the failed areas of base. The base can be re-stabilised with cement and bituminous emulsion or milled out and replaced with BTB if economically viable.
- Extensive crack sealing is done on the rest of the pavement with or without the use of membrane strengthening for active or wider cracks.
- Excessive rutting can be fixed by using a coarse slurry or one of the proprietary quick set emulsion products such as Colrut, Ralumac or similar.
- Do a proper drainage survey and replace non working or not existing subsoil drains. Many pavement failures can be attributed to poor drainage and, by correcting this, the pavement may eventually settle making further reconstruction unnecessary. In many cases the earth side drains are not deep enough or blocked and by opening and reshaping these, the drainage problems may be resolved.
- After all the failed areas have been fixed, the total section is re-surfaced using a single or double seal or asphalt, depending on the traffic, necessity of a pre-treatment, cost and time span envisaged till final rehabilitation.

5.3.2 Re-surfacing of pavement.

A pavement can be resurfaced for functional purposes or as part of a short term rehabilitation or preventative maintenance strategy. Also as part of the medium and long term rehabilitation options would re-surfacing be part of the process.

In Chapter 2, Par 2.3, Option 2, the various resurfacing options for various functional problems are mentioned. In Figure 5.3 a resurfacing selection guide is given by which the resurfacing option for a specific mode of surface failure can be determined. It is slightly amended from the diagram originally developed by Dr Steve Emery and associates.

When deciding on a resurfacing option the following must also be considered:
(Also refer to TRH3 [(11)])

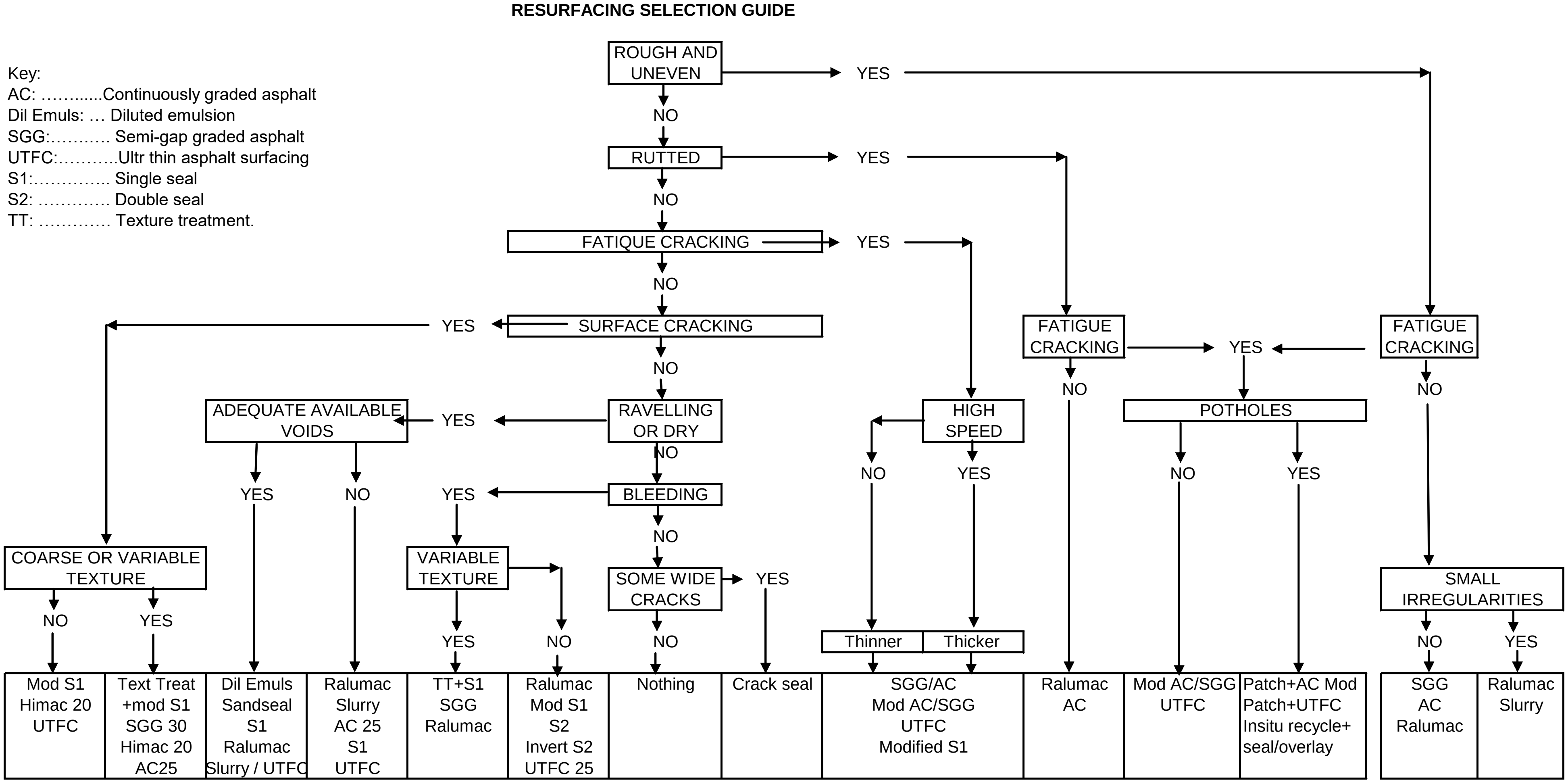
Traffic conditions. An unmodified seal may, under heavy traffic conditions, perhaps have a limited lifetime and modifying the binder with a polymer may considerably extend the lifetime of the seal. Under very heavy traffic conditions an asphalt overlay could be the better option.

Environmental conditions. Seal work can only successfully be done during summer time which limits the time per year for construction. If this is a problem, the application of an Ultra Thin Friction Course (UTFC) may be a viable alternative.

Texture of the existing pavement. For the successful application of a seal the texture depth of the existing pavement must not be more than 1,0mm. When it is more, pre-treatment with slurry is recommended. In such a case the application of an UTFC can also be considered because it may be more cost effective on the long run.

For practical considerations when designing and constructing the above, see section 6.

Figure 5.3 Resurfacing Selection Guide.



The diagram in Figure 5.3 can be used as a guide to determine the specific action to be taken to improve the functionality of a specific section of road.

5.4 Medium term rehabilitation options.

5.4.1 Basic principles

This rehabilitation option is used when enough funds are available to partly reconstruct sections of the road pavement but not the total pavement. In the pavement analysis discussed in Chapter 4, the remaining life of the pavement is calculated and, for each uniform section, a rehabilitation strategy decided upon to provide a predetermined service life. In the medium term the service life would be 10 to 15 years while, for long term rehabilitation, the service life would be 15 to 25 years. This is the rehabilitation strategy most often used for various reasons but mainly because the reconstruction of the total road would be much too expensive for most road authorities to consider.

The rehabilitation strategy for both medium and long term would be more or less the same. For the long term strategy the extend of the work would cover most of the pavement area whereas, for the medium term strategy, only selected areas will be rehabilitated to give more or less the same service life as for the un-rehabilitated sections.

For the medium term rehabilitation options the following actions would be appropriate:

- Do a proper pavement analysis using the procedures described in Paragraph 4.
- Decide on the design life of the rehabilitated sections compared to the remaining life of the sections of road not to be rehabilitated.
- Determine uniform sections and design the pavement structures for these sections to give the required design life.
- Do any of the rehabilitation actions as described further on to satisfy the design requirements.

In our example, it is evident from the analysis, that for certain sections of road the problem lies in the base and subbase. Also, the asphalt surfacing in the slow lane was shown to be fatigued with virtually no remaining structural life.
The choice of rehabilitation options would, however, not only depend on the so-called failed areas but also on the funds available for rehabilitation of a specific section of road. Especially when deeper rehabilitation is required, could this option be quite expensive. The various options are discussed below.

5.4.2 Reconstruction or addition of structural layers

When a design and analysis software program such as Rubicon Toolbox® is used, the decision on what layers of a pavement structure have to be reconstructed and how is made much easier. The existing structure in the analysis is replaced with the properties of the new structure and the analysis re-run to find the ultimate solution. Because changing properties of layers, thickness of layers etc. is quick and easy, the analysis can be done in a short period of time. If such a program is not available, the problem is much more complicated and a manual analysis must then be done on a few selected spots by using the principles discussed in Appendix C.

For the analysis of the pavement structure a decision has to be made of how many additional equivalent E80's or years of service must be provided for, based on the current and expected traffic growth. The various options are then analysed and changed until the required demand is satisfied. Such an analysis is shown in Figures 5.1 and 5.2. In Figure 5.1 an analysis was done using Rubicon Toolbox on an existing pavement structure. The existing top 250mm (except for the asphalt) is then re-stabilised with cement, giving the structure in Figure 5.2. The difference in expected life of the pavement is 4,1 million standard axles or approximately 9 years based on the expected traffic loading.

Another option is to be conservative and make use of previous experience and expert knowledge and use the deflection data and deflection indices to indicate areas of potential problems and then decide on a rehabilitation strategy. For that purpose the catalogue designs in TRH4(4) can also be considered.

In practice, the following options should be considered:

Option 1: Reconstruct some pavement layers by stabilising or replacing them.
Option 2: Add an additional layer(s) on top,
Option 3: Mill and replace a certain depth of the pavement with an asphalt base.

All these variables can be modelled and the best and most economic option chosen. In decision making the following must be considered:

Option 1 is the most expensive and would disrupt traffic for a while when the in-situ layers are re-constructed. It must also be decided what the stabilising agent should be as well as the final specified requirements. Recently more use is made of deep recycling using a deep milling/ recycling machine. It is also important to decide on the width of the treatment. When a single carriageway road is considered, reconstructing the one lane at a time and, if necessary, then the other lane, would allow traffic to use one half of the road while the other half is being reconstructed. On dual carriageway roads, the problem is easier as the slow lane is usually indicating failure long before the fast lane. That allows for the slow lane to be reconstructed while traffic is accommodated on the fast lane.
The width of treatment decided upon must be practical. To reduce the cost one would obviously try and make the reconstructed area as narrow as practically possible. Experience has shown that a four metre wide strip is usually sufficient placed in the lane such that the joint does not fall on an expected wheel path. This is further discussed in 5.7.

Option 2 where additional layers are added on top is sometimes the best option as one does not loose the integrity of the existing layers. This option is not practical when only sections of road has to be reconstructed as it will cause a height difference in the different sections. Also, one must evaluate the cost of the following when constructing a layer on top:

- The total road width will have to be reconstructed.
- New shoulders will have to be constructed due to the increased edge drop.
- Side drains and guard rails will have to be replaced.
- Over-bridges could have a clearance problem.
- All turn-offs and/or ramps will also have to be lifted.

Option 3 where existing base is replaced with asphalt base is definitely the quickest and easiest but not necessarily the cheapest option. If the analysis indicates that say, a 100mm asphalt base replacement for the existing base, will give adequate extra life to the reconstructed pavement, and the areas of reconstruction are relatively small, this would be the best option.

A proper economic analysis of the various options will have to be done to decide on the most financial viable one within the allocated budget.

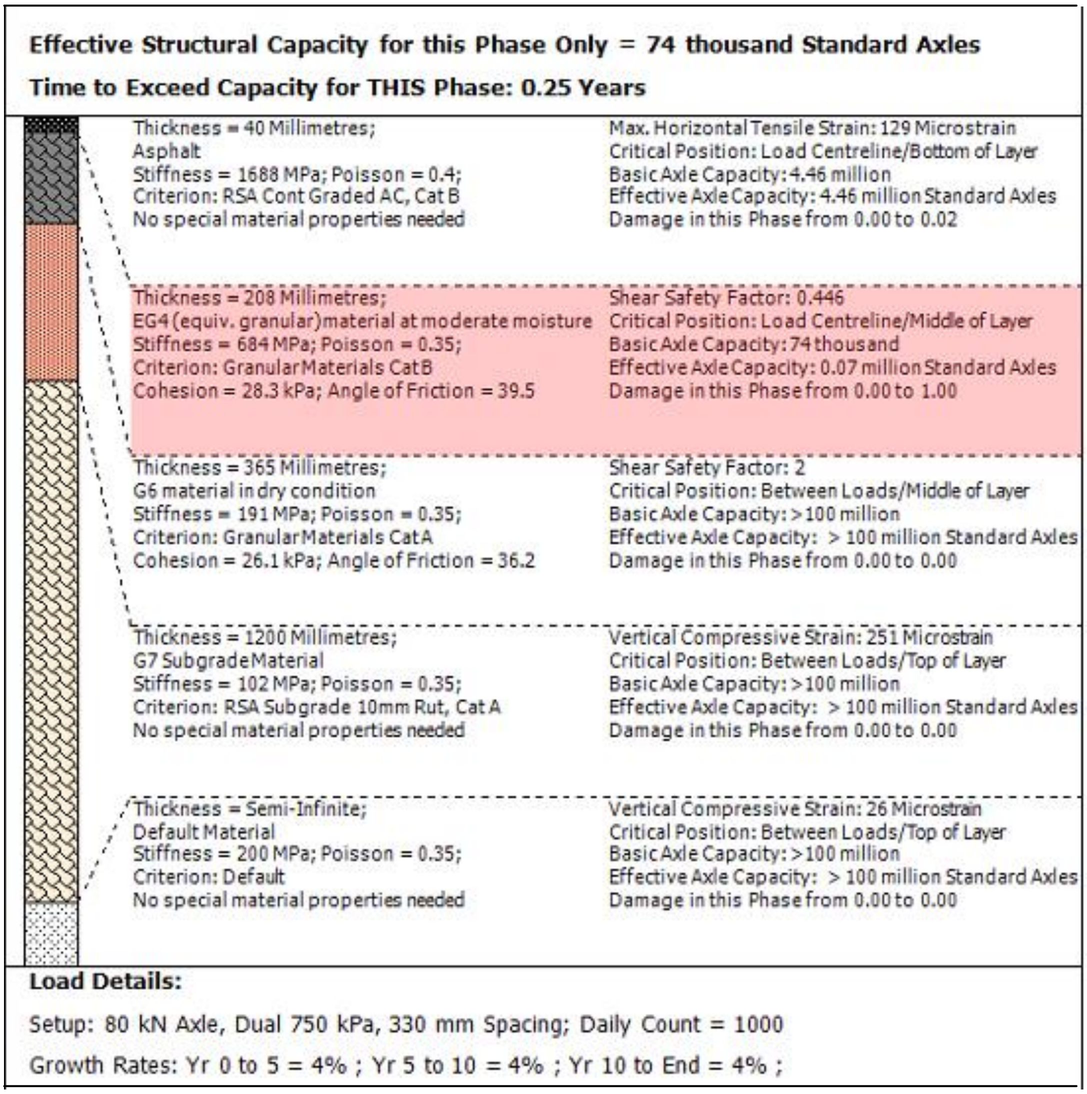


Figure 5.1: Structural capacity evaluation of the original pavement

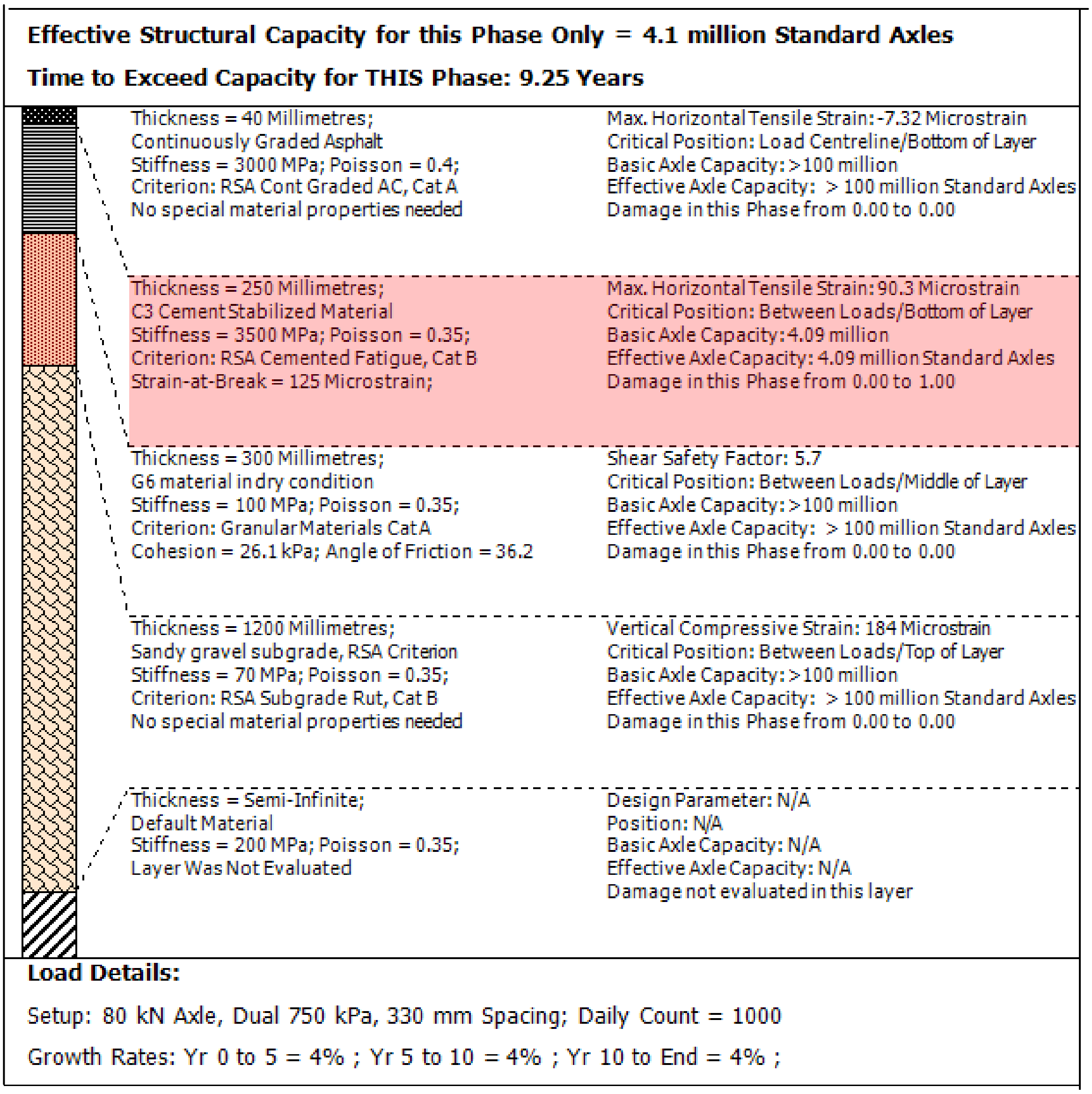


Figure 5.2: Structural capacity evaluation of amended pavement structure.

5.5 Long term rehabilitation options

The long term rehabilitation options exactly the same procedure as for the medium term options are followed except that in the design of the pavement structure an extended service life of 20-25 years is used. When this is done it would be found that most of the pavement structure will have to be reconstructed or strengthened. For the long term rehabilitation option one would also perhaps rather opt for an asphalt surface as the wearing course in stead of a seal.

5.6 Economic comparison and selection of rehabilitation options.

After the pavement analysis has been done, the various options available to construct the new rehabilitated pavement must be considered. Some smaller amendments to the design can also be tested to see what the influence on the design life would be.
In our example, for instance, the asphalt layer can be reduced to 35mm and various options for the stabilised layer can be investigated.

Options to be consider in an economic analysis would be aspects such as:

For base and subbase:

- Un-stabilised base with stabilised subbase layers used as an overlay on the existing pavement,
- Recycle existing layers and stabilise with cement or lime
- Recycle existing layers and stabilise with bituminous emulsion and cement
- As above, stabilised with foamed bitumen.
- Mill and replace base or part of it with ETB or BTB.
- Mill and replace asphalt surfacing

For surfacing:

- Asphalt of various thicknesses, modified or unmodified
- Asphalt with reinforcement
- Double seal using straight run bitumen or modified binders.
- Cape seal
- Ultra-thin friction coarse.
- Stone mastic asphalt
- Pre-treatment of existing surface with slurry seal before sealing.
- Single seal straight or modified binder.
- Pre-treatment of existing surface with rejuvenator (consider various types)
- Pre-treatment of existing surface with proprietary products (such as SP2000)

Ancillary works:

- Sub-soil drainage
- Surface drainage

When additional layers are added:

- Rebuilt shoulders
- Replace guard rails
- Replace concrete side drains.

Life cycle cost analysis of rehabilitation options:

The calculation of the life-cycle cost of a rehabilitation option is necessary if a proper analysis of rehabilitation options has to be done. Some of the inputs to such an analysis is, unfortunately, not always known or accessible to the designer and is, therefore, mostly not done. A computer programme called REACT was developed many years ago to accompany the previous TRH12 document on pavement rehabilitation. The intention is that it will again form part of the new TRH12 document currently being revised.

In such a life cycle cost analysis the following should be considered during the expected life of the pavement for each rehabilitation strategy:

Cost to the road agency (present worth of cost):

- Construction cost
- Maintenance cost
- Salvage value
- Road user cost
- Probability of failure, including pavement structure, environmental conditions, traffic loading, design variables, etc.
- Implications of deferred maintenance (if nothing is done)

5.7 Practical design considerations.

A few practical design considerations are discussed that should be considered when doing a rehabilitation design.

5.7.1 Seals, overlays, surface treatments.

When considering the options to resurface the road to enhance it's functional properties, the following must be considered:

- When the average deflections of the existing pavement is fairly high, say in the order of 600μm, polymer modification of the binder in the seal or asphalt should be considered to prevent premature cracking.
- When the texture depth of the existing surface is more than 1mm, pre-treatment of the surface with slurry seal is recommended. The slurry seal must be trafficked for at least six weeks before seal work is commenced or until the ball penetration value (TMH6 method ST4) remains constant over a period of at least two weeks.
- When a texture slurry has to be applied and later a double seal, the total cost including double traffic accommodation, establishment, etc. should be compared with the cost of constructing an Ultra Thin Friction Course (UTFC) that would give a much better riding quality and longer life.
- When an overlay is placed, it is obligatory that the underlying layer is cleaned of all loose material such as sand and debris and to apply sufficient tack coat of bituminous emulsion to the layer to prevent slipping of the overlay, especially on bleeding (rich in binder) areas.
- Badly cracked areas can be overlaid using a reinforcing mesh or geotextile to assist in spreading the load over the badly cracked area. This option is an alternative to mill and replace. See the TG3 manual from the Asphalt academy for more information and design criteria.

5.7.2 When recycling areas of the pavement.

In the comparison of options, one must be consider what can be done or must be done on the areas of pavement not included in the rehabilitation action. If part of the pavement is to be deep recycled, the following options can be considered for the rest of the pavement:

Bring the rehabilitated area to the same level as the existing surface by including the existing surface material in the milled material of the base/subbase. In this case the total road width can then be either sealed or surfaced with asphalt.

Mill out the existing surfacing material, if it is asphalt and thick enough, and construct the new base/ subbase to a level lower than the existing surface and place new asphalt in the lower area to the level of the existing road. If this option is used, the total road area can be sealed with a single, double or cape seal or surfaced with asphalt or UTFC. The existing surface should be milled out approximately 200 mm wider on either side of the deep recycled area to create a bench to serve as a level for the recycled layers and to aid in load transfer.

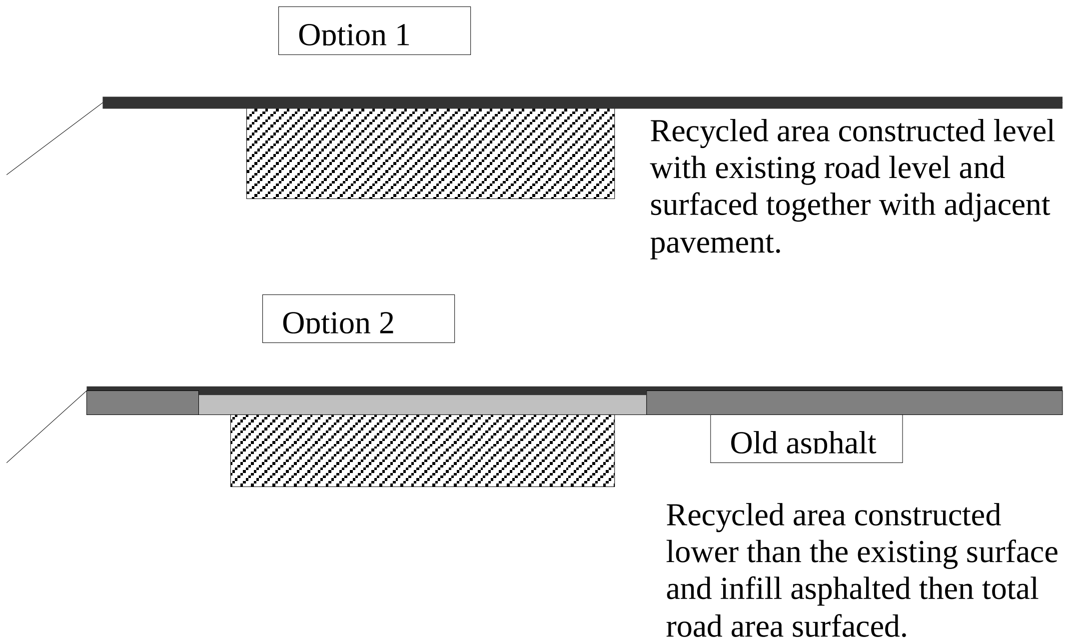


Figure 5.3: Options for deep recycling.

The width of the recycled area should be such that the construction joint is not below or too near a wheel track as benching in is not viable with deep recycling. It must also be recognised that heavy vehicles most often make use of part of the shoulder, especially on single carriageway roads and that the construction joint is placed well inside the shoulder if it is not included in the rehabilitation strip.

When option 2 in the above diagram is chosen and the existing base is crushed stone, it tends to be very friable when the surfacing has been removed. Care must be taken to prevent damage to that strip of base as far as possible.

When a recycling machine is used and the strip to be recycled requires more than one pass of the recycler, the mixed material tends to compact under the wheel of the recycler running on the previously mixed material during the second pass. To prevent uneven compaction and levelling, this compacted wheel track must be loosened by ripping using a grader before final levelling and compaction.

When fairly narrow strips are chosen for recycling, it is best to have a recycler that can run outside the area with only the pug mill (rotovator) inside the area to ensure that mixing is done right to the edge of the area. Some recyclers cannot mill below the wheel level.
The recycler must be heavy enough to handle the mass of recycled material without lifting up, especially when more than one pass of a recycler is required in a narrow area. A rotovator is thus not suitable for this purpose

When stabilising with bituminous emulsion and cement, a particle filter should be placed in the emulsion feed line to the recycler to prevent blocked nozzles.

When stabilising using a recycler the water quantity should be carefully regulated as it is virtually impossible to get rid of wet spots in the pavement in such a narrow area. Leaking pipes and valves should also be fixed as soon as possible to prevent wet spots. It must be remembered that, when cement stabilising, the moisture content should be 1-2% below optimum moisture content to prevent cracking (75-80% of saturation moisture content).

Compaction with modern vibratory and other compactors up to 300mm or even deeper is possible but the type of compactors and sequence must be carefully chosen for the specific material. Compaction must be done right up to the edge of the layer without damaging the sides of the adjacent pavement. The compaction moisture content must be well adapted to the compactors being used. From the specifications given in Paragraph 7 it will be seen that different compacted densities are usually specified for the top and bottom half of the compacted layer to ensure good compaction. These costs must be included in the estimate.

5.7.3 When widening the shoulders or adding lanes to an existing pavement.

Widening of an existing road by the addition of shoulders or an extra lane is often part of a rehabilitation project. The following aspects should, inter alia be considered.

When constructing any layer next to another, it is important that the existing layer is benched sufficiently to allow load transfer at the joint. The benching distance inside the layer must be more or less the same as the layer thickness. For thick layers of more than 300mm thick, two benches should be formed of 150mm each down to approximately 600mm. Below that depth is normally regarded as fill layers and the benches could be 500 mm high and at least 250mm wide.

The pavement design of the added on lane or shoulder should be such that a water trap is not formed. If the existing pavement has a granular base that may hold water, the added on lane or shoulder should be the same. If a stabilised base shoulder, that normally does not hold water and is fairly impermeable, is added, water may be trapped in the inside lane leading to premature failure on the joint.

The surfacing joint should not be in the same position as the base joint to prevent water ingress and provide a slightly better load transfer over the joint although thin surfacing layers may not contribute much to load transfer.

It is better to remove approximately 200mm of the old surfacing at the joint of the two layers for two reasons namely:
1 To give a starting level for the new layer to be constructed on the inside and
2 To form a new surfacing joint away from the base joint for the reasons mentioned above.

5.7.4 The addition of layers to an existing road

Extra layers can be added to an existing road either as a rehabilitation measure but also as part of a stage construction program.
Raising the level of the road would obviously have the disadvantage that other items such as guard rails, side drains, sign posts, bridge widths and over-bridge clearances will have to be checked and rectified. The shoulder width could also be a problem and will, depending on

how much the level of the road is raised, have to be widened. Traffic accommodation is also difficult. Preferably the total road width should be done in one operation but in difficult terrain and because of the extra costs involved in constructing deviations, half of the existing roadway and shoulder can be used for one-way traffic accommodation using a stop/go system. The aggregate overspill on the construction side always makes it difficult and dangerous for traffic to use it and special precautions should be taken to prevent overspill as much as possible. The road must be swept regularly to remove material next to the raised layer.

The following should, inter alia, be considered:

Before an extra layer is placed on an existing surfaced road, it is advisable to rip the existing surfacing. It will prevent a shear plane forming in the pavement system and also will not prevent moisture movement through the layers by creating a moisture trap.

If more than one layer is added, the layers must be benched on the centreline to prevent a crack from forming there.

The surfacing of the lane that is constructed first must stop short of the joint edge by approximately 200mm to allow an overlap of the surfacing with the base joint.

On curves with a cross-fall towards the side constructed first, water tends to accumulate against the constructed layers. This must somehow be prevented or the water removed as soon as possible to prevent wetting the already constructed layers.

5.7.5 Working in urban areas

Urban areas and especially city streets presents a number of unique challenges that has to be dealt with and of which the cost has to be considered in the budgeting process. Some of the issues that have to be considered are:

Traffic accommodation:
The fact that traffic and especially commercial traffic should preferably be delayed as little as possible places limitations on the type of rehabilitation that can be effected on city streets. The rehabilitation options should, therefore, be carefully selected with this in mind.

Overlays:
Due to the restrictions placed on overlay thickness due to kerbs, street crossings, accesses, pedestrian crossings, manholes, etc overlays can usually only be constructed once or perhaps twice on city streets and then only in fairly thin layers without any structural benefit to the pavement. The only way to re-surface city streets after this initial overlays would be to mill and replace the existing surfacing. During the mill- and- replace action, the use of high modulus asphalt or similar high stiffness layers should be considered to replace existing layers.

Services:
Some services are not always properly marked on drawings and not always at the expected depth. This could pose problems with recycling existing pavement layers to any depth of more than 300mm. This can be overcome by designing the replacement or recycled layers such that the load bearing capacity would be increased to cater for the shallower depth of recycling.

Working around manholes and similar obstructions also makes the recycling and mill and replace work more difficult. These aspects should be brought under the attention of the contractor by providing for it in the Schedule of Quantities. Special attention should be given to compacting layer works or asphalt surfacing around these obstacles. Buried manhole covers, pipes etc within the working depth may also damage recycler or miller teeth if their position is unknown.

Accesses:
Accesses to properties and especially commercial properties creates problems of their own. Especially when reconstructing the pavement it may happen that access to shops etc may be closed for a period of time. The one solution would be to consider doing this type of work overnight or using an asphalt base to replace existing layers in lieu of more conventional means of construction. This would obviously have a cost implication that has to be considered in budgeting.

Induction loops.
Many crossings with traffic lights have induction loops to initiate light changes and for law enforcement purposes. These loops are only a few mm below the road surface and if mill and replace or recycling is to be done, they will be destroyed. This can be overcome by having the traffic department on standby to immediately replace the loops after the new surfacing has been constructed. Traffic lights can be placed on a regular time related switching mode until the loops can be replaced.

5.7.6 Sub-soil drainage

Water is the main enemy of a road pavement and many of them fail due to a lack of proper subsoil drainage. Part of the initial assessment should, therefore, include an evaluation of the condition of existing sub-soil and side drainage as well as the necessity for new drainage where pavement failures due to excess water is evident.
Most subsoil drains are installed beneath the concrete side drains when the road is initially constructed. To replace them in the same position is, therefore, virtually impossible. One alternative is to place the new drain on the outside of the concrete side drain, As most side drains are in cuttings this is, however, not always possible. In such a case the alternative would be to place the new drain on the inside of the side drain using a ditch-witch or similar excavator. In Chapter 7 special provisions are given to cover this type of construction.

It should, however be considered, where possible, to install a traditional subsoil drain as these fin drains are quite expensive and not always functional.

It has been found that many of the so-called fin drains eventually stop functioning due to the geofabric, covering the drain, blocking. One option is to remove the geofabric and fill the ditch with 9,5mm single size stone around the skeleton of the drain. The slots in the drain are 5mm, preventing the aggregate to go into the drain itself.

6 MATERIALS DESIGN

6.1 Introduction

After the pavement structure has been designed and the materials properties for each layer to be rehabilitated established, the material for each individual layers has to be designed.
For our example, as indicated in Figure 5.2, the layers of the rehabilitated pavement should be as follows:

40mm of Continuously graded asphalt
250 mm C3 Stabilised material.
300mm G6 in-situ gravel, etc.

- The G6 in-situ material is the same as the original pavement and can, therefore, be left in place. As mentioned earlier, the 40mm existing asphalt is badly fatigued and will have to be removed or milled in with the existing base and subbase material. As it will have to be replaced with new asphalt, it was decided, in this case, to mill and remove the existing asphalt over the total length of the slow lane, 4,5 m wide. (Option 2 in paragraph 5.7.2)
- The base and subbase material will be deep recycled to a depth of 250mm and 4,0m wide, stabilised with bituminous emulsion and cement up to the level of the old base.
- New asphalt modified with Sasobit, to increase the softening point of the binder and to prevent rutting, will be paved over the new recycled areas. It will also be paved to replace the milled out slow lane asphalt, level with the existing surface.
- The total area will be re-sealed using a 16,0 and 6,7 mm double seal with SBS modified 80/100 pen bitumen.

Each of the materials used in the pavement has to be designed to comply with the specified requirements.

In the following paragraphs the design of the various materials, including the above, will be discussed.

6.2 Crack sealing

Before any resurfacing can be placed on an existing pavement all cracks larger than 3mm must be sealed. Cracks are normally sealed using a polymer modified sealant either hot applied or in emulsion form. Active cracks should be properly cleaned by hot air and preferably sealed by hot sealant. In APPENDIX E more details on the choice of sealant and methodology of crack sealing is given.

6.3 Surface enrichment sprays.

Surface enrichment sprays are used to rejuvenate existing asphalt or sealed surfaces and thereby extend their service life. This is more a preventative maintenance tool but could also form part of a rehabilitation strategy. More details on this application is given in APPENDIX F.

6.4 Asphalt

Asphalt mix design is done using the new South African Hot mix Asphalt (HMA) design procedure presented in Reference 12, 13 and 14 and it will not be discussed in detail here.

Consideration should, however, be given to using a modified binder in the HMA. Especially on high volume roads, this has been shown to extend the service life of the asphalt considerably especially to prevent rutting and fatigue cracking.

6.5 Ultra-thin Asphalt Friction Courses (UTFC)

UTFC usually is a patented open graded asphalt layers using polymer modified bitumen. It is very similar to Stone Mastic Asphalt or the so-called popcorn or open graded asphalt mix given in the Colto Specification (Table 4202/9). Being patented, the design as well as construction control and quality is governed by the patent. For that reason a performance specification is used to govern it. Such a specification is presented in Chapter 7.

6.6 Surfacing seals

The most frequently used seals are Single, Double or Cape seals with normal bitumen or polymer modified binder. In some instances cut back bitumen and bitumen emulsions can also be used and the design adapted.
The designs of surfacing seals are given in TRH3 (11).

6.7 Cementitiously stabilised material

The mix design principles for cementitiously stabilised materials is given in Appendix G.
The choice of cement to be used shall depend on the availability and type of construction. It must be realised that, when using a recycler for mixing in the stabiliser, a quicker setting cement can be used than when mixing is done by grader and plough. It must be realised that the additives in blended cements do not really take part in the cementation reaction for stabilisation and the use of blended cements are not recommended – again, depending on availability. The use of CEMII-32,5 AS or AV is recommended. For more information and design and construction details the SARF course on stabilisation can be attended.

6.8 Bituminous stabilised materials

The mix designs for materials stabilised with bituminous materials such as bitumen emulsion, bitumen emulsion with cementitious stabiliser and foam bitumen are given in various documents. The TG2 document on Bitumen Stabilised Materials, published by the Asphalt academy is the most recent and authoritative document on the subject.
It is also included in the SARF course on stabilisation.

6.9 Reinforced asphalt overlays.

When asphalt reinforcement with geogrids or similar materials is indicated, the design guideline in the TG3 manual by The Asphalt Academy can be used. Refer to reference 16.

7 CONSTRUCTION

7.1 Introduction

The intention in this section is not to give detailed procedures for each type of rehabilitation construction because that is contained in such documents as the COLTO Standard Specifications [9] and recommendations in other SANRAL documents such as TRH3 [11]. The focus would rather be on special provisions and tips for constructing the various options. These tips are not exhaustive or obligatory and would not be applicable under all circumstances. The logical sequence of clauses in the COLTO Standard Specifications is followed. As in most Project Specifications the prefix "B" is used to indicate an amended clause.

As Environmental issues and the Occupational Health and Safety (OH&S) Act are becoming more important, some clauses are given to cover those aspects. It is accepted that each road authority would have his own OH&S document and it is referred to in this document. These aspects are more fully discussed in Paragraph 8.

7.2 SECTION 1300: Contractors Establishment on site and General Obligations.

B 1302 GENERAL REQUIREMENTS

(a) Camps, constructional plant and testing facilities

Add the following:

"No camp site may be established inside the road reserve although material may be stockpiled inside the road reserve if previously arranged with the Engineer and approved by the client. The contractor must provide suitable portable toilet facilities that are regularly serviced at the construction site. Payment for these facilities shall be included in the contractor's establishment cost."

(c) Legal and contractual requirements and responsibility to the public

Add the following:

"All costs incurred to comply with the requirements of the Occupational Health and Safety act of 1993 as well as additional requirements as specified in the Client's Safety Health and Environmental document, shall be included in the fixed obligations tendered by the contractor.

When the activities of the contractor cause damage to vehicles of the travelling public, directly or indirectly, such as damage caused by stone loss on seal work, the contractor shall handle and pay all claims emanating from such incidents and endeavour to settle all reasonable claims in an amicable way. If the contractor fails to settle such claim, the Employer reserves the right to settle such claim on behalf of the Contractor and deduct such costs from the monthly payment certificates. Such payments are provided for in the Schedule of Quantities."
Add the following sub-clause:

"(d) Compliance with the Occupational Health and Safety Act and Environmental Management plan.

The contractor shall in all aspects comply with the requirements of the Occupational Health and Safety act (Act 85 1993, latest edition) as well as the Client's requirements for Health and Safety and Environmental Management Plan".

B 1303 PAYMENT

Delete item 13.01 (b): "Value related obligations" and the relevant paragraphs referring to this item. Value related obligations are not payable and all of the contractor's general obligations must be included in item 13.01 (c).

Add the following:

"<u>Item</u>		<u>Unit</u>
B 13.02	**Provisional sum for the settlement of monetary claims received from the travelling public for vehicle damage caused by the contractors construction activities.**	**Rand**

Payment shall be a monetary deduction to the value of all individual claims received from the public and not settled by the Contractor to the value of the agreed settlement amount between the claimant and the Employer and shall be entered as a negative amount in the schedule of quantities in the contractor's monthly payment certificates."

Add the following item:

"<u>Item</u>		<u>Unit</u>
B 13.03	**Payment for compliance with the specified Occupational Health and Safety and Environmental requirements.**	
(a)	**Fixed obligations**	**Lump sum**
(b)	**Time related obligations**	**Month**

The tendered lump sum shall be for implementing the specified Occupational Health and Safety requirements and Environmental Management Plan including all required appointments, training of personnel, personal protective equipment (PPE), etc. required in the relevant documents. The tendered rate for time related obligations shall be for the provision of a dedicated Occupational Health and Safety and Environmental control officer, maintaining PPE and continuous training and audits as specified in the relevant documents."

7.3 CLAUSE 1500: Accommodation of Traffic.

Traffic accommodation during rehabilitation work is most often quite problematic because one has to work under traffic. If at all possible, detours or deviations can be used but this could be quite expensive and in difficult topographical terrain, impossible. The main objective should, however, be to disrupt the normal flow of traffic as little as possible. The following general guidelines can be applied.

To enforce compliance with the traffic accommodation requirements, it has been found useful to include the following provision:

B1503 TEMPORARY TRAFFIC-CONTROL FACILITIES

Add to the second paragraph:
"The contractor will be penalised for non-compliance with the requirements of traffic accommodation with a fixed penalty of R5 000 per incident of non-conformance. A time related penalty of R500 per hour will be applied over and above the fixed penalty if defects are not rectified within the time frame, which shall not be unreasonable, set by the supervisory Engineer or the Client's Health and Safety Officer. If the contractor fails to conform to the instruction timely, the time related penalty will apply from the time the instruction is given to the contractor to the time at which the instruction was satisfactorily executed."

In most Contract Specifications, the separate items for traffic accommodation is listed and paid for. It has been found useful to make Traffic accommodation a lump sum item and that the contractor decide on his own destiny and calculate his own risk, especially on rehabilitation contracts where especially delineators are regularly destroyed if not cared for by the contractor. Also, detailed drawings are usually provided for traffic accommodation whereas all the relevant drawings and specifications are found in the SA Road Traffic Signs Manual, Chapter 13. The idea is to stop spoon feeding contractors and let them take responsibility for their own actions.

Add to the third paragraph:
"Traffic signs and placing shall be in accordance with the relevant paragraphs and drawings in the newest addition of the South African Road Traffic Signs Manual, Chapter 13: Road works signing, section 13.9 (0r 13.11 for freeways). The contractor shall obtain his own copy of the manual and shall have a copy available on site at all times. The contractor must note that although traffic cones is allowed in stead of delineators next to lane closures, in Chapter 13, that it is a requirement that a delineator shall be used at least between every three cones. At ramp exit and entrance areas only delineators shall be used together with the other specified signs."

(b) Road signs and barricades

Add to the first paragraph
"The contractor must, in addition to the specified temporary road signs for traffic accommodation, supply and erect at least six temporary 'NO LINES' warning signs of the appropriate size to be placed at the beginning and equally spaced through the construction section on both lanes during surfacing operations and before line marking has been done. Similarly, when the possibility exists that loose stones may damage vehicles; loose stone signs (TW338) shall be erected in advance of the problem areas. Similarly, where an asphalt surface is being constructed, surface step advance warning signs (TW340 or TW341) must be used. The cost of all these must be included in the tendered rate for traffic accommodation."

(e) Warning devices.

Add to this sub-clause:
"In addition to the specified Occupational Health and Safety requirements, all workers working on or near the road shall be equipped with the following: New clean

- **Yellow or orange overalls.**
- **A reflective safety jacket.**

The contractor must ensure that these items are always in a clean and good condition.

Supervisory staff, inspectors or visitors to the working area shall at all times wear a clean reflective safety jacket.

Any person found on site without an effective safety jacket shall immediately be removed from site."

B 1513 ACCOMMODATION OF TRAFFIC WHERE THE ROAD IS CONSTRUCTED IN HALF WIDTHS

Add to the first paragraph:
"The maximum continuous length of road that shall be closed for traffic accommodation in one direction is limited to 4 km. The length of area that can be worked in one direction at any given time is limited to 2 sections of 3 km with a minimum of 4 km fully open to traffic before a next section of 3 km can be worked. **Sections where road marking has not been done shall be regarded as being under construction and not fully opened.**

The number of sections under construction at any given time and in any direction shall not exceed three."

Delete the fourth paragraph starting with: "The number of sections…"

B 1517 MEASUREMENT AND PAYMENT.

Amend item 15.01 to read as follows:

"Item		Unit
B 15.01	**The provision of temporary traffic control facilities (except traffic lights and amber flicker lights for the supervisory staff).**	**Lump sum**

The unit of measurement shall be a lump sum, payable as follows: 40% of the tendered amount with the first certificate and the balance in equal monthly instalments, for the provision of temporary traffic accommodation and control facilities over the length of road on which such control facilities has to be provided irrespective of the number of lanes used by traffic or it being single or double carriageway roads, width of the road or the number of road areas on which the contractor elects to work at any one time or the number of relocations of such facilities during the contract period. It shall also include the length of road where construction is done in half widths.

The tendered rate shall include full compensation for accommodating traffic on the existing road, or diversions alongside work areas or where the road is constructed in half widths as detailed in the drawings including the provision of a traffic safety officer, adequate road signs, delineators, traffic cones, plant, labour, transport, communication equipment, amber rotating lights, safety clothing, etc. required to install, maintain, relocate and monitor traffic control facilities as specified and to the satisfaction of the Engineer. The rate shall **exclude** the provision of traffic lights where the road is constructed in half widths and amber rotating lights for use by the supervisory staff."

Add the following item:

"Item		Unit
B15.14	**Penalty to be deducted for non-compliance with accommodation of traffic requirements.**	
	(a) Fixed penalty per occurrence:	Number (No)
	(b) Time related penalty	Hour (h)

The unit of measurement shall be a fixed penalty sum of R5000 per occurrence for each occurrence for which a non-conformance is raised in respect of traffic accommodation with an over and above penalty of R500 per hour for the time since the raising of the non-conformance to the time the defect has been rectified when the rectification of the defect is not done within the time limit set in the non-conformance.
The amount of the penalty shall be deducted in the monthly payment certificate."

7.4 SECTION 2100: Drains

As mentioned in 3.4 and 5.7.6, inadequacies in drainage should be corrected during the rehabilitation project. Two types of corrective action can be instituted namely, The cleaning and deepening of earth side-drains and the installation of subsoil drains. Blocked or non functional subsurface drains would usually have been installed below existing concrete lined side drains and are difficult to open and repair. It has been found that it is most often less expensive to abandon these drains and construct a new drain on the outside of the concrete side drain.

In the following example project specifications, these drains are replaced with a composite drainage system but it can just as well be replaced with the standard drainage system normally preferred by the specific road authority if sufficient space is available. The prerequisite is that the water table must be drawn down deep enough to prevent water seeping into the pavement layers of the road. In the example, some lengths of drain is installed inside the concrete lined side drain due to a stone cutting and thus lack of space on the outside. Provision is also made for the outlet of the drain in existing drainage structures.

B 210 4 SUBSOIL DRAINAGE

(b) Construction of subsoil drainage system

(iv) Composite in-plain drainage system

Add to this sub-sub-clause:

"Where composite in-plain drainage systems are to be installed behind existing concrete side drains or in the pavement next to the concrete side drain, it shall be in-plane composite flow net type as indicated on the drawings. The digging of the trench, installation, backfilling etc. shall be done without damaging or disturbing the existing concrete side drains. All damage to these drains shall be rectified at the cost of the contractor.
Some subsoil drainage is to be installed within the road pavement next to the concrete side drain where the pavement material is to be in-situ stabilised. It is advisable that the installation of the drains is done before the recycling commences. In such a case, trenches shall be temporarily backfilled in such a manner that it is safe for traffic. The drains may also be installed during the recycling operation if it will not influence the specified completion times for the stabilisation operation.
Outlets to subsurface drains may be either cast in-situ concrete units as indicated on the drawings or by breaking into the wing walls or side walls of existing culverts, catchpits or manholes. Where such a catch pit is in an existing concrete side drain and the side drain has to be partly demolished for installation of the outlet pipe, the foundation for the side drain shall be properly re-compacted and the side drain repaired to the satisfaction of the engineer."

B2106 MEASUREMENT AND PAYMENT.

Amend item 21.12 as follows:

"Item		Unit
B 21.12	(a) Concrete outlet structures for subsoil drainage systems complete as specified.	Number (No)
	(b) Breaking into existing drainage structures to form outlets for subsoil drains.	Number (No)

The unit of measurement and payment for (a) shall be as for item 21.12 specified in the Standard Specifications.
The unit of measurement for item (b) shall be the number of holes broken into the concrete walls of existing drainage structures including removing of concrete in existing side drains where required.
The tendered rate shall include full compensation for all materials, plant, tools, labour and supervision to make holes of the required size into the walls of existing drainage structures to accommodate the outlet pipes of subsoil drainage systems including the placement of the pipe, cost and installation of the specified wire mesh over the opening, repair of over-breaks, backfilling, the re-instatement and repair to side drains, etc."

Provision is made in the COLTO Standard Specification, Clause 2102 and pay item 21.02 for the clearing and shaping of existing open drains.

7.5 Recycling existing pavement layers.

Recycling of existing pavement layers is recently becoming more popular and is, in most instances, the obvious choice for extending the service life of a pavement. Various options exist for the recycling of the existing layers, with deep recycling using a recycler being the easiest, most cost effective and most frequently used procedure.
Traditional construction procedures can, however, be successfully used if the width of road being recycled are such that a grader can fit into the area. It is often not the case and trying to mix stabiliser etc. in a narrow width with a grader, ploughs, etc. is just not worth the effort.
Removing the material and then mixing it elsewhere with stabiliser is an option but would most probably be much more expensive that in-situ mixing using a recycler. In addition, the pug mill type mixing done by the recycler is much more effective than mixing by traditional means.

The design of the recycled pavement would also determine what procedure should be the best to use. If only the base is recycled traditional means could be used. If, however, the base and subbase or even deeper layers are included then the use of a recycling machine would be better. Usually when pavement layers are recycled they are stabilised with any of the following:

Cementitious stabilisers such as lime and cement.
Anionic Bitumen emulsion with cement (as a cementitious stabiliser)
Bitumen emulsion with cement (as a bituminous stabiliser)
Foam bitumen.
Proprietary stabilising agents.

Some of the following project specification clauses can be useful:

SECTION 3400: PAVEMENT LAYERS OF GRAVEL MATERIAL

B 3401 SCOPE

Add the following:
"The specifications for the recycling and stabilisation of pavement layers are given under Section 3500 of these specifications."

B3402 MATERIALS

(a) General

Replace paragraph two with the following:

"Only in-situ material shall be used for recycling the pavement layers"

(b) Compaction requirements

Replace the compaction requirements for subbase layer with the following:

"The 250mm stabilised subbase layer shall have a minimum average compacted density of 96% of mod. AASHTO density. The lower 100mm shall have a minimum average density of 95% of mod. AASHTO density."

B3406 QUALITY OF MATERIALS AND WORKMENSHIP

Section 8300 will be used for the assessment of quality.

SECTION 3500: STABILISATION

B 3502 MATERIALS.

(a) Material for bituminous stabilisation

(i) Bituminous stabilising agents
The bituminous stabilising agent shall be 60% anionic bitumen emulsion.

(ii) Fillers
The filler shall be ordinary Portland cement – Cem. II-32,5 A-S or B-S cement.

SP3503 CHEMICAL STABILISATION.

(a) Preparing the layer

Replace the contents of this paragraph with the following:

"After the existing asphalt has been milled off, no further preparation of the layer is required."

(h) Curing the Stabilized Layer

The method of curing shall be sub-clause (iii) by applying a bitumen emulsion curing membrane (0,5ℓ/m² of 30% emulsion) immediately after final compaction of a day's work.

SP3505 BITUMINOUS STABILISATION.

(a) Preparing the material

Replace this paragraph with the following:

- Only 300m in length of total layer width may be recycled in one operation that includes spreading of the cement, mixing-in of cement with the addition of emulsion and water, levelling of the layer and compaction.

- The contractor shall provide for three sections of 300m to be done during a days work with the proviso that no new section shall be started after 15:00 to allow all actions to be timely completed before stopping for the day.

- No recycling shall commence or proceed when rain is eminent."

These conditions can be adjusted on site by the Engineer if the contractor proves that he can continuously maintain a higher production rate. "

(b) Mixing in the additive

Amend this paragraph to read as follows:

"The minimum prescribed quantity of cement shall be spread as described in clause 3503(b), taking clause B3505 (a) in consideration. The cement as well as water and bitumen emulsion shall then be mixed in, using a milling recycling machine as described in clause 3505(d)."

(c) Applying the stabilising agent

Replace the entire clause with the following:

"The cement plus emulsion and water required to reach the specified optimum moisture content of the material, shall be mixed into the existing base and subbase material to a minimum depth of 250mm using a suitable milling-recycling machine. The machine shall be capable of applying the bituminous stabiliser and compaction water from attached or separate delivery tanks in a continuous operation while milling/recycling the in-situ material.

The mixing speed of the machine shall be well controlled to properly mix the material without any visible clumping, balling or streaking of the mixed material to the full specified depth. The machine shall also be heavy enough to prevent it from being lifted up while the milling/recycling operation is being carried out.

In some instances where the asphalt layer is thicker than 40mm, the remaining asphalt shall be milled in with the crushed stone and cemented sub-base layer. No extra compensation shall be payable for this action.

The existing layer width shall be milled and stabiliser mixed-in in two or more parallel runs with a 300mm overlap between runs to the width as indicated on the drawings."

(d) Compaction

Delete the second and third paragraphs.

Add the following after the first paragraph:

"Before compaction commences, the layer shall be scarified to loosen the material compacted under the wheels of the recycling machine. It shall then be levelled to the required grade and compacted. Due to the narrow work area, the 250mm deep layer has to be compacted in one layer from the top. The contractor must provide sufficient heavy compaction plant to compact the layer to the required density. After compaction the surplus material must be removed by grading to the level tolerances specified. The surplus material removed during this operation shall be removed to stockpile or otherwise disposed of as directed by the Engineer."

(e) Construction limitations

No slushing shall be done. The provisions of the third paragraph shall apply.

B 3506 TOLERANCES

(a) Uniformity of mix (chemical stabilization)

Replace this sub-clause with the following:

"The uniformity of the stabilised mix shall be determined by doing Indirect Tensile Strength (ITS) determinations on randomly selected samples including different horizons of the stabilised layer and comparing the results. The maximum allowable difference in strength shall be 5%. The lowest strength obtained shall in all cases be higher than the specified minimum of X kPa or design strength for that layer".

B 3509 QUALITY OF MATERIAL AND WORKMANSHIP

Replace the second paragraph with the following:

"Section 8300 shall be used to measure test results and measurements.

B 3510 MEASUREMENT AND PAYMENT.

Amend item 35.07 as follows:

"Item		Unit
B 35.07	Bituminous stabilisation of existing pavement layers, 250mm deep using a recycling machine.	Cubic metre (m^3)

The unit of measurement shall be the cubic metre of material stabilised and compacted using a deep milling/recycling machine. The quantity shall be determined from the authorized dimensions of the completed layer.

The tendered rate shall include full compensation for milling the material to the specified depth, mixing in of the bituminous stabiliser, cementitious filler and compaction water, compacting the mixed material to the required density and levelling the completed layer to within the specified tolerances as well as the removal and disposal of excess material. It shall include all plant, personnel, tools and incidentals to construct the layer as specified in two runs with a 200mm overlap and removing and disposing of excess material within a free haul distance of 1km. No distinction shall be made in regard to the type of bituminous stabiliser or cementitious product used."

Add the following pay items:

"Item		Unit
B 35.14	Providing the recycling machine on site.	Number (No)

The unit of measurement shall be the number of recycling machines provided on site, or the number of times a milling machine is brought onto site where it had to be removed temporarily with the approval of the Engineer.

The tendered rate shall include full compensation for all costs involved in bringing the recycling machine on the site at the position where it has to start working in accordance with the contractor's approved programme. Payment for returning the machine to the site after removal shall only be made where the removal was within the contractor's approved program of work and for no other reason. Additional machines will only be paid for where provision is made for more than one machine in the contractor's programme as required to complete the work in the specified time frame and the employer or engineer approves thereof and the other machine(s) is fully utilized.

<u>Item</u>		<u>Unit</u>
B 35.15	Moving the recycling machine on site for a distance exceeding one kilometre.	Number (No)

The unit of measurement shall be the number of times the machine is moved on site for more than one kilometre as may be approved or instructed by the Engineer in writing.

The tendered rate shall include full compensation for all costs involved in such moving necessary for the execution of the work as well as all delays and production losses. Payment will not be made for purposes of maintenance, repairs or for replacement with another machine."

7.6 SECTION 4200: Asphalt base and surfacing

7.6.1 Normal asphalt

The following clauses have been found to be of value when smaller areas of asphalt have to be paved for rehabilitation. This does not apply to UTFC, which is covered in 7.6.2.

B 4202 MATERIALS

(a) Bituminous binder

(i) Conventional binders

The bitumen used for the asphalt base shall be 60/70 pen.

(iii) Homogeneous modified binders.

Add to this sub-sub-clause:

"The bitumen used in asphalt surfacing shall be a homogeneous modified binder using 60/70 penetration grade bitumen modified with SBS polymer to comply with the requirements for AE2 binder as specified in the TG1 manual.

(a) Aggregates

(viii) Grading

The grading of the combined aggregate including any filler added in an approved working mix shall be within the limits specified in Table 4202/7 for medium continuously graded asphalt for surfacing and in Table 4202/6 for the 26,5mm maximum size continuously graded base.

Only crushed rock and sand shall be used in any of the asphalt mixes specified. Natural sand shall only be used with approval from the employer."

(c) Fillers

For both the asphalt surfacing and base mixes, 1,0% of lime shall be added as filler without any additional payment. Filler shall be added inside drum type mixers as specified.

B 4203 COMPOSITION OF ASPHALT BASE AND SURFACING MIXTURES

Replace the fourth (4th) paragraph with the following:

For the continuously graded surfacing mix, 1,0% road lime by mass of total aggregate shall be added to the asphalt as active filler and the nominal mix proportions by mass for bitumen shall be 4,8% by mass. These requirements are for design purposes only and the actual rates and proportions used shall be determined to suit the materials and conditions prevailing during construction. Any design variations in binder and filler content will be paid for separately.

For the base mix, the nominal binder content for tender purposes shall be 4,5% by mass and 1,0% of road lime shall be added as active filler without any extra payment."

B4204 PLANT AND EQUIPMENT

(i) Conventional binders

Add the following paragraph below the sixth paragraph:

"For drum type mixers, the active filler shall not be added in the cold-feed line of the mixer but a separate feeding pipe shall be installed for feeding the required percentage of lime into the mix at the same position where the binder is fed into the mix."

B4205 GENERAL LIMITATIONS AND REQUIREMENTS AND STORAGE OF MIXED MATERIALS

(a) Surface requirements
(i) Tack coat

Add the following sentence:
"A tack coat shall in all instances be applied to the surface to be paved or covered with asphalt including all vertical cut edges. The tack coat shall be 30% cationic spray grade bitumen emulsion applied at 0,55 ℓ/m² and may be applied by hand sprayers. Tack coat may not be applied to the excavated area more than four hours before the asphalt is placed."

B 4215 MEASUREMENT AND PAYMENT

Add the following items:

"Item		**Rate**
B 42.21	Establishment of asphalt paving unit on site	
(a)	Initial establishment	Lump sum
(b)	Moving the paving unit on site for distances more than 1 km.	Number (No)

The unit of measurement for (b) shall be the number of times the paving unit is moved on site for a distance of more than 1km on agreement with the engineer.

The tendered rates shall be all inclusive and shall provide for the establishment of all plant, operators, personnel, labourers, tools, transport, etc. to establish the total paving unit consisting of the paver, rollers, haulage trucks, tools, etc on site or to move it from one location to another."

7.6.2 Ultra Thin Friction Courses (UTFC)

Because UTFC is normally a patented product, it cannot be governed by the standard specification items. In such a case a performance specification is given and the product is guaranteed for three years or more, as decided by the Employer. The designer or specifying agent should also provide for the money guarantee in his Provisions of Contract.
The following amended clauses can be used as basis for the compilation of such a performance specification.

B 4201 SCOPE.

(a) Add the following sub-sub paragraph:

"(viii) Ultra-thin Friction Course (UTFC). The asphalt shall be designed by the designer/ patent holder or concessionaire for the UTFC system and shall be mixed and paved by an approved franchise holder or owner of the system. The tenderer shall, with his tender, supply the Client with the following information on the system:

- The name and patent holder of the system,
- Documentation proving the successful implementation of the system in South-Africa as well as elsewhere on similar highly trafficked roads.
- Proof that the system can be successfully implemented by using locally available materials.

B 4202 MATERIALS

(a) Bituminous binders.

(iii) Add to this sub-sub-clause:
"The modified binder used in the UTFC shall be SBS modified 80/100 or 60/70 pen bitumen and shall comply with the requirements of type A-E2 binder specified in document TG1: ' The use of Modified Bituminous Binders in Road Construction, published by the Asphalt Academy.' All bituminous binders shall be transported and handled as recommended in SABITA manual 25: 'Quality management in the handling and transport of bituminous binders."

(b) Aggregates.

Add to this clause:
"Coarse and fine aggregate shall comply with the requirements set by the designers of the UTFC and the controls shall be set by them.

(c) Fillers.

Replace this clause with the following:
"Fillers used in the UTFC shall be as specified by the designers of the system and the controls shall be as set by them."

B 4203 COMPOSITION OF ASPHALT BASE AND SURFACING.

Add to this clause:
"The composition of the UTFC asphalt shall be as determined by the designers or patent holders of the system and shall in all aspects comply with their requirements.

B 4204 PLANT AND EQUIPMENT

(a) General

Add the following to this clause:
"The plant and equipment used in mixing, paving and compacting the UTFC shall be as specified by the designer/ patent holder of the system and shall in all aspects comply with their requirements."

B 4205 GENERAL LIMITATIONS AND REQUIREMENTS AND THE STORAGE OF MIXED MATERIAL.

Add the following to this clause:
"Limitations, requirements and storage requirements and limitations of mixed materials shall be as specified by the patent holder of the UTFC. Weather limitations shall be specified by the designer and shall be used on site for control purposes. "

(c) Surface requirements

(iii) Tack coat
Add to this sub-sub-clause:

"For the UTFC asphalt the cost of the tack coat shall be included in the asphalt price as it forms part of the patented procedure. Only the tack coat applied below other types of asphalt shall be applied and measure as specified in this clause."

B 4207 SPREADING THE MIXTURE

As special spray pavers are used to pave the propriety UTFC mixture, paving and compaction shall be done as specified by the patent holder /designer of the mix. Due to the high stiffness of these mixes hand work shall be limited as far as possible and, if done, shall be done while the mix is still at its specified paving temperature and in the shortest time possible. This precludes the paving of large areas by hand. As the paving contractor shall be aware of this restriction, an uneven surface finish will not be acceptable and shall comply with the requirements in clause SP4213.
If adjacent lanes are not constructed within one week during summer temperatures, or earlier during cold conditions with average ambient temperatures <20°C, longitudinal construction joints shall be sawn with a diamond pavement cutter to form a level joint. Joints between lanes shall be as straight as possible and, if found to be out of specification, it shall be sawn straight and the loss in paving material shall be for the cost of the contractor.

B 4210 COMPACTION
Compaction of the UTFC shall be as specified by the patent holder / designer of the system. If acceptable to the designer, suitable soluble oil may be used as lubricating agent on rollers.

B 4213 CONSTRUCTION TOLERANCES AND FINISH REQUIREMENTS

(a) Construction tolerances

The UTFC shall comply with the requirements for: "Other asphalt layers for freeways" in this clause, even if the work is not done on a freeway.

(f) Construction tolerances for overlays,

(iii). Replace paragraphs 3 and 4 with:

"The calculated average thickness of the overlay for each days production shall be the nominal specified thickness ±4mm. For the UTFC layer it shall be the nominal specified thickness ±2 mm. The average thickness per days production shall be determined from the volume of asphalt used divided by the paved area."

Add to paragraph 5: "For overlaid areas, the surface regularity, in addition to the specified requirements, shall be determined with a high speed profilometer (HSP), capable of producing a class 1 vertical measurement and class 3 longitudinal sampling distance as defined in ASTM standard E950-94, with a valid validation certificate. Record the longitudinal profile in both wheel tracks, 1,7m apart for each paved lane (including the shoulders if these are paved separately). Then, from the data, determine the average 100m IRI (International Roughness Index) values in units of mm/m for each wheel track and determine the average IRI for the left and right wheel track for each 100m section for each lane paved in one width.

For overlays placed on an existing surface as well as overlays placed on a newly constructed surface or new asphalt overlay, the average 100m riding quality values in terms of the IRI shall be judged in terms of the payment adjusting factors in Table B4213/2.

TABLE B4213/2: RIDING QUALITY FOR OVERLAYS PAYMENT ADJUSTMENT FACTOR.

Riding quality	**Payment adjustment factor**	
100mm IRI values (mm/m)	**Overlay on existing surface**	**Overlay on new surface or overlay**
<1,20	1,0	1,0
1,21-1,30	1,0	0,99
1,31-1,40	1,0	0,98
1,41-1,50	0,98	0,96
1,51-1,60	0,97	0,94
1,61-1,70	0,96	0,92
1,71-1,80	0,94	0,90
1,81-1,90	0,92	Not acceptable
1,91-2,0	0,90	Not acceptable
>2,0	Not acceptable	Not acceptable

Sections that is found to be unacceptable in terms of regularity shall either be replaced or remedial measures implemented that will provide the required riding quality. A method statement describing these measures shall be approved by the Employer before implementation. Corrective work shall be done at the Contractor's expense. After completion of the corrective work the specific 100m section shall be re-evaluated as described above.

Any adjustment to the payment of asphalt surfacing shall be done by multiplying the payment adjustment factor derived as above with the full payment of the relevant asphalt pay item plus tack coat, pre-coated chips and other payable incidentals.. The payment adjustment factor shall apply to the full layer width paved in one operation for that specific 100m section.

Riding quality tests using the HSP shall be paid for under pay-item B42/81.02. See clause B8115.

(c) Gradings
Add to this clause:
"For the UTFC, the combined aggregate and filler grading shall comply with the requirements specified by the designer of the UTFC system."

(d) Binder content
Add to this clause:
"For the UTFC, the binder content and tolerances shall comply with the requirements specified by the designer of the UTFC system."

(e) Voids
The contents of this clause shall not be applicable to the UTFC.

B 4214 QUALITY OF MATERIALS AND WORKMANHIP

Add sub-sub clause (d):
"(d) Performance guarantee.

(i) Introduction
The performance of the UTFC shall be guaranteed by the patent holder /concessionaire /contractor for defects in materials and workmanship for a period of at least three years after construction and the contractor must ensure that all relevant routine inspections and test are performed as specified by the designers of the system to service the guarantee.

(ii) Assessment of performance
The aspects below will be judged by the end of each year for three years following the issuing of the Taking-over Certificate in the sole discretion of the Employer. The inspection shall be done by the Employer together with the contractor/ franchise holder and designer.

1. Deformation (shoving)
2. Surface failure (de-bonding)
3. Surface cracking (excluding reflective cracks)
4. Surface ravelling (Aggregate loss)
5. Bleeding.
6. Surface regularity.

The visually assessed parameters shall be judged in terms of the definitions for degree and extent as described in TMH9. Each lane and shoulder shall be individually assessed for segment lengths of 1 km.

For each of the assessed parameters the Combined Index Value (CIV) is calculated as follows:

$$CIV = \sum_{Degree=1}^{5} Degree \times \frac{(Length_{Degree})}{1000}$$

$$= \left(1 \times \frac{Length_1}{1000}\right) + \left(2 \times \frac{Length_2}{1000}\right) + \left(3 \times \frac{Length_3}{1000}\right) + \left(4 \times \frac{Length_4}{1000}\right) + \left(5 \times \frac{Length_5}{1000}\right)$$

Where $Length_i$ = Total linear length (in metres) of degree i over the 1km segment, etc.

The contractor shall be responsible to report on the findings of the visual survey and shall deliver his report within 30 days after the inspection to the employer together with remedial measures proposed to deal with areas that do not comply with the acceptance criteria in table B 4214/1.

(iii) Acceptance criteria.

Table B 4214/1: Acceptance criteria for visual inspection of UTFC.

Type of distress	Time (years)[1]	Maximum allowable	
		Degree[2]	CIV[3]
Deformation (shoving) of asphalt	1	1	0,0
	2	2	0,2
	3	2	0,3
Surface failure (deboning)	1	1	0,0
	2	2	0,1
	3	2	0,2
Surface cracking	1	1	1,0
	2	2	2,0
	3	2	3,0
Surface ravelling	1	1	0,0
	2	2	1,0
	3	3	3,0
Bleeding	1	1	0,0
	2	2	1,0
	3	2	3,0

Notes:

1 Time in years after completion of the works.

2 The degree of the visually assessed parameter in accordance with TRH9: "Pavement management system: Standard Visual assessment Manual for Flexible Pavements" issued by COLTO (1992)

3 Combined Index Value (CIV) as calculated above per one kilometre length of lane or shoulder.

(iv) Acceptance and rejection

The following procedures shall be followed to determine the acceptance of the UTFC surfacing after each assessment done by the Engineer and the Client at the specified time.
After each assessment, the Engineer shall advise the contractor of defects evident during the inspection and may either:

- Instruct the contractor to undertake immediate remedial work
- Grant a concession to allow the remedial work to be held in abeyance until further notices are issued.
- Notwithstanding the above point, the contractor may elect to attend to the defects immediately.

Should all parameters meet the full Acceptance Criteria three years after the taking-over certificate has been issued, the contract shall then be completed and all retention monies (guarantees) can be released and the contractor would have no further liability for the performance of the work except as provided for in clause 52.3 of the General Conditions of Contract.

(v) Remedial work

Where distress has occurred in excess of the permissible or maximum specified limits relating to the performance of the pavement surfacing, the contractor shall, at his own cost and in terms of the Contract, rectify all segments of the pavement in which such defects occur.
The Engineer's and Employer's approval shall be obtained prior to any remedial work being carried out by the contractor and he shall commence with the remedial work within thirty (30) days from the date the site is handed over to him for such remedial work. A programme of work shall be submitted for approval to the Employer or his agent within fourteen (14) days of the date of such notification.
In the event of the contractor's failing to undertake the remedial work as required to reinstate the surface to an acceptable condition to conform to the specified requirements, the Employer reserves the right to withhold and /or apply the payment of such monies due to the contractor or that may become due to him in accordance with the contract.

The following shall apply for all remedial work or repairs done by the Contractor:

1. The Contractor shall, at his own cost, supply, erect and maintain the necessary traffic accommodation signs, personnel, etc. referred to in Section 1500 of the contract.
2. The Contractor shall, at his own cost, repair/ reinstate any items such as road studs, road markings, etc. damaged or influenced by the required remedial work.
3. When the requirements in Table B4214/1 as well as in Clause 4213 have not been met, the various types of remedial work contained in Table B4214/2, depending on the degree of distress, shall be undertaken by the Contractor at his own cost to reinstate or rectify the specific defects in the pavement surfacing.
4. The contractor will be liable to reimburse the Employer for any supervision fees and other supervision costs paid and incurred by the Employer for the time the Contractor

is on site.

5. Final acceptance shall only be given to the Contractor upon completion of all remedial work to the satisfaction of the Engineer and employer.

Table B4214/2 Remedial work for UTFC surfacing.

Type of Distress	Minimum remedial measures
Deformation (shoving) of asphalt	Mill out full depth of defective surfacing and repave.
Surface failure	Mill out full depth of defective surfacing and repave.
Surface cracking	Seal cracks or mill out full depth of defective surfacing and repave.
Surface ravelling	Repair using UTFC surfacing mix or mill out full depth of defective surfacing and repave.
Bleeding	Mill out full depth of defective surfacing and repave.
Surface regularity (Clause 4213 (a) (v))	Mill out full depth of defective surfacing and repave.

B 4215 MEASUREMENT AND PAYMENT

Replace item 42.02 with the following item:

"Item		Unit
B 42.02	**Asphalt surfacing (state type of bitumen)**	
(c)	Ultra-thin asphalt surfacing as specified, 25mm thick.	Ton (t)

Add the following to the tendered rate paragraph:

"The tendered rate for the Ultra-thin asphalt surfacing shall include full compensation for all materials as specified including the emulsion tack coat and additives to the binder, as well as the performance guarantee."

7.7. Resurfacing using chip seals

It is recommended that for the design and construction of seals the recommendations in the COLTO specifications [9] and TRH 3 [11] is studied. The following additional special provisions can be considered:

Some contractors have not enough plant available for complying with the requirements set in COLTO. To ensure the availability of sufficient plant, a penalty can be imposed for the non-availability or of specific plant items. A schedule can be provided, listing the plant and priced by the contractor as if for day-works. The following special provision can then be included:

Clause 4303: Plant and equipment:
"(l) Penalty for the non-availability of specified plant
If any item of plant specified for use in the sealing operations in not on site or in-operational for whatever reason, the Engineer may, at his own discretion, apply a penalty for that specific item of plant at the rate tendered in Section 1800: Dayworks* of the tender documents plus 15%. The amount of the penalty shall be entered as a negative amount in the Schedule of quantities for interim payments.

Add under clause 4315:

"Item		Unit
B 43.02	Penalty for non-availability of specified plant	
	List plant items from (a) to (z)	Rand

The unit of measurement shall be the rate tendered for the specific plant item in Section 1800* for the time unit as tendered and for the total time the specific item of plant was not available for performing it's task as specified during seal operations, calculated for the period of time that such operations were carried out.

Payment shall be a monetary deduction to the value of each individual plant item not available for sealing operations as specified plus 15%."

*(If used, a day-works schedule listing all plant etc shall be included in the Bill of Quantities)

Clause B 4307: Construction of seal.

(b) Single and double seals
(ii) Initial rolling of the aggregate

Replace the first (1st) sentence with: "Rolling shall commence immediately after the application of aggregate has started. It is imperative that rolling shall take place as close as possible behind the chip spreader. If the strip being done at a time is so long that the roller falls behind with more than 50m, a second or more rollers shall be employed to limit the distance as far as possible."

(iii) Broom drag and final rolling of aggregate

Add the following at the end of the first (1) paragraph:

"The contractor shall provide a back-chipping team and pneumatic-tyre roller with sufficient capacity to complete the back-chipping and rolling operation within the one hour time period allowed for below."

Replace the third (3rd) paragraph with the following:

"Final rolling of the seal, before the fog spray has been applied, shall be done by at least four passes of a 15-20 ton pneumatic-tyre roller followed by two passes of a 6-8 ton steel wheeled roller or as decided by the engineer. If the aggregate tends to crush under the roller, a lighter roller shall be used or, on instruction from the Engineer, the use of the steel wheel roller shall be terminated. All rolling operations shall be completed within one hour after starting. If the area that has been sealed is such that rolling could not be completed in time, extra rollers shall be employed.

After the fog spray has been applied and had enough time to dry, the seal shall again be rolled with at least four passes of the pneumatic-tyre roller where-after it shall be properly broomed before opening to traffic."

(iv) Protection of kerbs, channels, etc.:

Add to this paragraph:
"The cost of protecting other road elements from being soiled by binder shall be included in the cost of the sealing operation."

Add the following sub-sub clause:
"(vi) Final brooming and cleaning.

After the seal has been finally rolled and the binder had sufficient time to set, before opening to traffic, all loose aggregate shall be removed from the road surface by rotary broom. Brooming shall be done such that the broomed aggregate does not stick to the over-sprayed strip of binder next to the sealed strip. When opened to traffic and further aggregate are loosened, the road shall again be broomed to remove the loose aggregate.

All broomed off and waste aggregate, paper strips, etc. next to the road shall be picked up as soon as possible but not later than 48 hours after opening to traffic to prevent loose stones from being carried onto the road surface by traffic etc. Excess aggregate and other waste products shall not be discarded onto the side slopes of the road or in the road reserve but shall be disposed of in an environmentally friendly manner.
The cost of brooming and cleaning shall not be paid for separately and shall be included in the rate for seal work."

B 4308 RATES OF APPLICATION.

Add to the first paragraph:
"The hot application rate of the binder as specified is taken as 1.08 times the cold application rate and this factor shall be used for determining the cold application rate for payment purposes from the specified hot application rate."

Add to the fourth paragraph:
"The aggregate application rate shall be pre-determined and agreed upon by the contractor and engineer from a dry-run of the chip spreader, using a 1x1m canvas patch. This application rate shall then be applied for payment purposes. A variation

on the application rate will only be allowed when an increase or decrease of the application rate is instructed by the Engineer and pre-determined as above."

B 4311 OPENING TO TRAFFIC

Add to the second paragraph:
"For double seals the road shall not be opened to traffic until the final fog spray has been applied to the second layer of stone and had sufficient time to dry to such an extent that it is not being picked up by traffic and has been rolled and broomed as specified in clause 4307."

8 OCCUPATIONAL HEALTH, SAFETY AND ENVIRONMENTAL ISSUES

8.1 Occupational Health and Safety

The road authority is obliged to demand total compliance to the Occupational Health and Safety Act (Act 85 0f 1993). Complying with this act will result in contractors incurring expenses and for that reason it is only fair to be compensated for it. This is covered under Section 1300 of the standard Specifications (See Clause 13.03 under paragraph 7.2).
The following obligations have to be imposed on contractors:

1 The Client has a duty, in terms Construction Regulation 4, to stop unsafe work or work in unsafe conditions and that any loss of production that may result from the execution of this duty, or from any stoppage of work by an inspector of the Department of labour or by any other government official or body will be carried by the contractor. The contractor must guarantee that such costs will be included in his tendered rates.

3. The contractor must take particular note of the duties imposed on the contractor and principal contractor by Construction Regulation 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27 and 28 and make provision for any costs that may be implied by compliance therewith in the contract sum.

4. The contractor will be expected to utilise a formal risk assessment programme throughout the course of the project. This programme will identify hazards, evaluate the inherent risk factors and determine the appropriate precautionary measures. The contractor takes note of his statutory obligation to issue Personal Protective Equipment (PPE) to employees only where no better precautionary measure is available (section 8(2)b).

5. The contractor shall issue PPE and replacement PPE and any other type of safety equipment free of charge to his employees, and shall make provision for this expense in the contract sum. Any unforeseen cost incurred in this regard will be carried by the contractor.

6. The contractor takes note of the fact that the Client reserves the right to instruct him to obtain and issue his employees any PPE or safety equipment that it deems appropriate and the cost for such additional equipment will be carried by the contractor.

7. The contractor takes note of the fact that the Client has a regulatory duty to audit the works at a frequency that will be agreed upon, but at least once a month, as per Construction Regulation 4(d).

8. The contractor shall provide the Client with a copy of his internal Health and Safety manual of procedures and standards, as well as confirmation of the registration of all his employees with the Workmen's Compensation Commission and a certified copy of a Letter of Good Standing from the commissioner.

9. The contractor will be provided with the safety specifications of the Client that will be applicable to this project [as per CR 4(1){a}] and will subsequently provide The client

with a suitable and sufficiently documented safety plan based on the specifications [as per CR 5(1) and CR 5(4)].
In addition, the contractor shall keep a health and safety file as per CR 5(7).

10. The contractor shall provide the Client with all the information required. The information referred to is the health and safety selection criteria that will be used during the selection process. All information provided shall be accurate and true, the information given will be verified and false information will lead to automatic disqualification.

11 The Contractor shall appoint competent persons as per Section 16(2) of the OHS Act. Any such appointed person shall be trained on any occupational health and safety matter and the OHS Act provisions pertinent to the work that is to be performed under his responsibility. Copies of any appointments made by the Contractor shall immediately be provided to the Client.

The Contractor shall further ensure that all his employees are trained on the health and safety aspects relating to the work and that they understand the hazards associated with such work being carried out on the premises. Without derogating from the foregoing, the Contractor shall, in particular, ensure that all his users or operators of any materials, plant, machinery or equipment are properly trained in the use of such materials, plant, machinery or equipment.

12 The Contractor shall ensure that he has an updated copy of the OHS Act on site at all times and that this is accessible to his appointed responsible persons and employees.

13 The Contractor shall implement safe work practices as prescribed by the Client and shall ensure that his responsible persons and employees are made conversant with and adhere to such safe work practices.
The Contractor shall ensure that work for which a permit is required by the Client is not performed by his employees prior to the obtaining of such a permit.

14 If required in terms of the OHS Act, the Contractor shall establish his own health and safety committee(s) and ensure that his employees, being the committee members, hold health and safety meetings as often as may be required and at least once every three (3) months. The Client may elect to permit the Contractor’s health and safety representatives to attend the Client’s health and safety committee meetings.

15 The Client observes a strict accident prevention policy, which includes the investigation of all accidents or incidents involving injury to people, or damage to property, to identify causes to permit the introduction of controls and measures to prevent recurrences.

The Contractor shall ensure that his employees, as well as the employees of his agents, sub-contractors and service providers, co-operate fully with any accident or incident investigation.

The contents of this provision shall in no way relieve the Contractor of his legal obligation to report certain incidents to the Inspector of the Department of Labour as defined in the Act, and to keep the records required in terms of the Act.

All incidents referred to in Section 24 of the OHS Act shall be reported by the Contractor to the Department of Labour and to The Client. The Client shall further be provided with copies of any written documentation relating to any incident.

16 The Contractor shall ensure that all his employees undergo routine medical examinations and that they are medically fit for the purposes of the work they are to perform.

The Contractor shall ensure that the work site and surrounding area is at all times maintained to a reasonably practicable level of hygiene and cleanliness. In this regard, no loose materials shall be left lying about unnecessarily and the work site shall be cleared of waste material regularly and on completion of the work.

Consistent with the requirements of the Agreement and these Rules, the Contractor shall ensure that all areas to which his agents, sub-contractors and service providers are assigned or perform work are kept neat, clean, and orderly.

17 No intoxicating substance of any form shall be allowed on site. Any person suspected of being intoxicated shall not be allowed on the site. Any person required to take medication shall notify the relevant responsible person thereof, as well as the potential side effects of the medication.

18 The Contractor shall ensure that all road vehicles used on the premises are in a roadworthy condition and are licensed and insured. All drivers shall have relevant and valid driving licences and, where applicable valid Professional Driving Permits (PDP), and no vehicle shall carry passengers unless it is specifically designed to do so. All drivers shall adhere to the speed limits and road signs on the premises at all times.

In the event that any hazardous substances are to be transported on the premises, the Contractor shall ensure that the requirements of the Hazardous Chemical Substances Act 15 of 1973 are complied with at all times.

8.2 Environmental issues

Any Client or contractor involved in rehabilitation works on roads should have an Environmental Management Policy (EMP) that has to be adhered to during rehabilitation works. The intension is not to provide here a complete EMP but merely to highlight the most important aspects. Some of the aspects given below are enforceable by law and others are to ensure good housekeeping by the contractor.

The following aspects need to be focused on in the EMP:

- Soil Management and Borrow Pits.

  Mining permits and rehabilitation plans.

- Waste Management and Control.

  Handling and disposal of solid and liquid waste.

- Vegetation Management and mowing of grass.

  Protection of listed plants.

- Noise Management.

  Especially important in or near urban areas.
- Air Quality Management.

  Excessive dust from road works and haul vehicles, smoke from asphalt plants, etc.
- Water Management.

  Protection of water course, prevention of contamination or rivers and ground water.
- Fire Management.

  Fire breaks around camps. No open fires next to road.
- Fencing Management.

  Repair of damaged fences, fencing of hazardous areas.
- Emergency Management.

  Traffic accidents, Hazardous spills, etc.
- Establishment of temporary contractor camps.

  Sanitation, water, waste management, housekeeping
- Cultural heritage and/or archaeological elements.

  Protection of cultural and archaeological areas, artefacts, graves, etc.
- Social aspects – safety of the public.

  Traffic safety, crime due to camps, etc.
- Environmental Permits. The following may be required:

  Water use license for any River bed/bank diversion or disturbance,

  Water use license for any change, alteration or extension to any bridge or culvert,

  Permit for bulk fuel storage on site,

  Permit for storage of hazardous material including bitumen and pre-coated chips,

  Permit for any alterations or demolition of any structure older than 60 years,

  Mining permit for any borrow pits or quarries,

  Permit for an Asphalt Plant.

  Permits must always be obtained before any work is started.

- Specific Environmental Issues.

  Servicing of vehicles and oils spillage

  Informal traders and camp-followers creating social problems

- Spoiling of waste material
- Stockpiles.

    Choose areas with smallest impact on environment.

    Positioned and sloped to create the least visual impact.

    Constructed and maintained so as to avoid erosion of the material and contamination of the surrounding environment.

    Kept free from all alien/undesirable vegetation

    Reinstate to original condition, replant grass, etc.

- Environmental Monitoring and Auditing.

    Auditing by the Engineer will serve to assess the following:

    The implementation of the plan in full;

    The assessment of the effectiveness of mitigation measures;

    The implementation of recommended corrective actions;

    The effectiveness of communication and record keeping.

The Engineer must ensure and monitor the implementation of these management steps. On an overall framework, the effectiveness of all environmental management measures will have to be monitored and audited on a regular basis.

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APPENDIX A

Method for Consistent Classification of Material for Pavement Rehabilitation Design.

Excerpts from Appendix A in the: Manual TG2 for Bitumen Stabilised Materials by the Asphalt Academy (Ref 15).

A.1. INTRODUCTION

The derivation of the material classes for each pavement layer is a critical aspect of the design process, since this process effectively constitutes the determination of structural design inputs. When appropriately done, the classification process also provides a link between the mix design and structural design processes. In this Appendix, the recommended method for classifying materials is presented. The object of the method is to provide a reliable, rational and consistent indication of the appropriate material class. The method is based on the use of all available information, and uses fuzzy logic and certainty theory to assess the certainty that materials belong to a particular class.

The sections below describe the method in more detail and provide all relevant details for the implementation of the method. Although the method was specifically developed for use in the structural design method for pavements that incorporate BSMs, the approach can be used in any pavement design context with all common material types, and is especially relevant for rehabilitation design.

A.2. CONCEPT

During a routine pavement rehabilitation investigation, an engineer is typically faced with a wide array of test parameters and condition indicators. These parameters can be quantitative or qualitative, subjectively or objectively determined, and the sample sizes for different indicator types may vary significantly. For example, for a specific pavement layer within a uniform design section, an engineer may typically be faced with the following set of information:

Seven Dynamic Cone Penetrometer (DCP) tests.
Fifty Falling Weight Deflectometer (FWD) deflections.
Two sets of material descriptions and samples from test pits, together with standard materials test results, including Plasticity Index (PI), grading, California Bearing Ratio (CBR), moisture content and density.
One subjective visual assessment with a description of observed distresses.
Fifty semi-subjectively determined back-calculated stiffnesses from FWD tests.
A general description of the material type from historical records.
A general description of the history and past performance of the pavement.

From such a set of information, the engineer has to derive the key assumptions needed for the rehabilitation design. The synthesis of the information to arrive at design assumptions is one of the most important and difficult parts of the rehabilitation design process.

Apart from basic analytical skill, it also requires considerable experience and knowledge of the main drivers of material behaviour. When incorrectly done, inconsistent conclusions can be drawn and the design assumptions will not be consistently supported by available evidence.

The concept behind the material classification method is therefore to guide engineers in the interpretation of available pavement condition data, and to synthesize available information so that key design assumptions can be derived in a consistent and rational manner. The objective of the materials classification method is therefore to provide a method for the consistent assessment of pavement materials using routine tests and indicators.

Many material classification methods are specification type approaches that rely on pass or fail type criteria. For these type of approaches, if any one test fails the criteria for the material class then the material cannot be classified as that class. For example, if the CBR value is below the specification for

a G6 material, then the material cannot be classified as a G6 even if all other available test results do meet the G6 criteria.
The approach described in this Appendix is a more rational, albeit less exact method, which can handle vagueness in the data. Rather than giving a yes or no answer, the method indicates the conformance to a material class in less restrictive terms. The approach assesses the certainty that the material can be considered as the particular material class, and uses Fuzzy Logic to provide this type of assessment.
The evaluation of pavement materials as part of rehabilitation investigations poses several unique challenges. These challenges are related to the realities of pavement investigations and pavement design, which includes the following aspects:

Many Sources of Uncertainty: Pavement engineering deals with large quantities (i.e. long distances) of natural and thus highly variable materials, which are subject to highly variable loads.
Risk is Poorly Defined: The risk associated with pavement failure is poorly defined. For example, what is the consequence of 5% more crocodile cracking over a twelve year period, and is it cost-effective to spend an extra R10 million now to prevent it? Several assumptions are needed to answer this question, and many of these – although they can be estimated – are beyond control (e.g. rainfall, overloading, future budgets, etc). Because of this situation, subjective assessment using experience plays a considerable role in pavement design.
Small Sample Sizes: Reliance on small samples is part of the reality of pavement investigations. It is not unusual for a rehabilitation design over 20 km of road, over varying terrain and geological areas to be based on ten or less trial pits.
All Tests are Indicators: In pavement design situations, the assessment of materials always aims to assess stability and (for some materials) flexibility. It does so either directly (as in a stability test) or indirectly (as in a grading assessment, which will impact on stability). Because the actual load situation varies, no pavement material test is able to completely quantify long term stability or flexibility. Even a highly sophisticated test, like the repeated load triaxial test, must be performed at a fixed moisture content and stress state which will never correspond completely to the real pavement situation that it aims to assess. Thus all tests provide only a relative indication of the two key properties to be assessed, and some tests do so very poorly.
Interpretation is Vague: In pavement rehabilitation investigations, an engineer needs to decide what information is available, and what can be done with it. A yes or no interpretation is not always appropriate, and a relative interpretation is needed. This complicates the interpretation of data considerably, especially when conflicting information is involved. It also introduces more subjectivity into the process.

The material classification system deals with these realities. Specifically, the approach incorporates the following elements:
Clear and rational formulation of the objective.
Ability to handle vagueness and uncertainty of interpretation.
Ability to work with small sample sizes.

A.2.1. Assumed Material Behaviour

To provide a sound basis for the materials classification method, a model of pavement material behaviour was adopted. The assumed model is shown in Figure A.1, and represents the material as a conglomerate of course particles, fine particles, bitumen and air voids. This material model is the well-known Mohr-Coulomb model, which generally applies to composite materials consisting of a combination of loose aggregate and bitumen. This model applies to almost all pavement engineering materials except clay and silt and manufactured materials such as geotextiles, with the important distinction that the composition of the mastic differs significantly for different materials

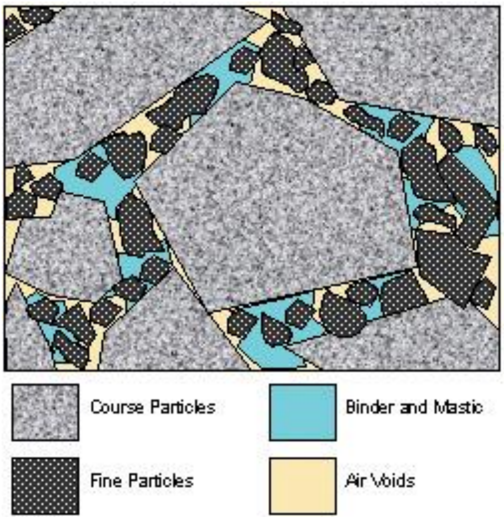

Figure A.1 Mohr-Coulomb Material Model

The material model shown in Figure A.1 can be used to explain the components that determine the strength and stiffness of the material. There are two components that determine the material's shear strength:

The cohesive strength, which is determined mainly by the mastic (consisting of the mixed bitumen and fine material), and **The strength provided by inter-particle friction**, and mobilized when compressive stresses force the fine and coarse particles together. The cohesive and frictional strength components determine not only the shear strength or stability, but also the stiffness and tensile strength. When the material is in compression, the stiffness and shear strength is primarily determined by a combination of the cohesive and frictional elements. When the material works in tension, particles are not pushed together and the stiffness and tensile strength are determined mainly by the cohesive element (i.e. the mastic). Stability and flexibility provide an indication of the resistance to the two main sources of pavement deterioration: deformation (either due to volume change or shear) and cracking (due to fatigue in tension).

The materials that are most resistant to shear and tensile failure are those in which there is a good balance between the strength provided by the cohesive and frictional elements. However, some materials tend to be dominated either by the frictional or cohesive element, as illustrated in Figure A.2.

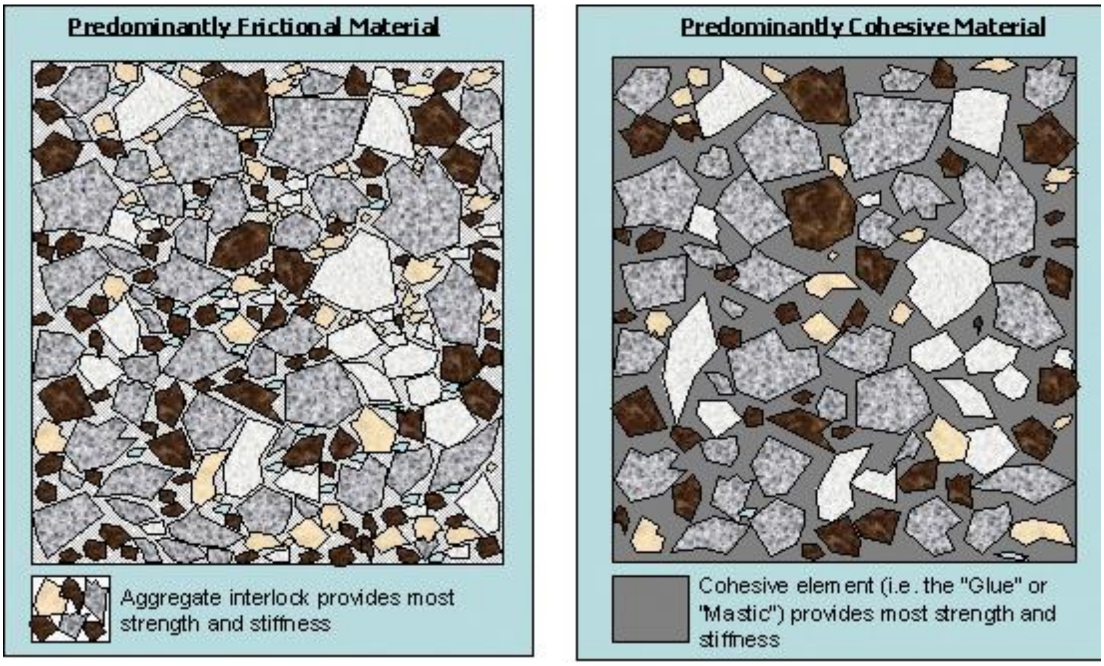

Figure A.2 Material Composition, Showing Dominance of Friction and Cohesion

The relative role that the frictional or cohesive component plays in determining the strength and stiffness depends almost entirely on the state of the mastic. In the case of asphalt, for example, the mastic consists of the bitumen and filler combination. At high temperatures, the visco-elastic bitumen softens. When the material is loaded in this condition, load is transferred directly to the coarse aggregate matrix, and shear strength is almost completely from the frictional component. A similar effect is observed in crushed stone and natural gravels, where excess water destroys the suction forces that bind the fine particles together in a mastic, thereby significantly reducing the cohesive strength or stiffness component.

A clear understanding of the role of cohesion and friction in determining the strength and stiffness of pavement materials is important, as most test indicators provide an assessment of one or both of these elements. Thus some tests, like the Plasticity Index (PI), relate only to the cohesive element, while others, like a grading analysis, relate to the shear strength element. A fundamental understanding of what is measured by a specific test can provide the key to a rational and useful interpretation of the test's results.

The above definition and discussion of the Mohr-Coulomb model, and the cohesive and frictional components that drive this model, are used in Section A.4 to classify the various materials tests, and to guide their interpretation

A.2.2. Design Equivalent Material Class

The material classes for granular and cement stabilised materials adopted for this material classification method are aligned with TRH14 (1985). The TRH14 classification system is regarded as being highly suitable for new construction and rehabilitation design, as the behaviour and performance patterns of each material class is known with some certainty. However, with the material classification method, the obtained materials class is regarded as the design equivalent materials class (DEMAC) and will not necessarily meet the specifications for that material class as given in TRH14. However, since materials to which design equivalent classes are assigned have been in service for some time, the raw material would conform to (or exceed) the specifications for the class, as stated in TRH14, in almost all instances. The material classes for BSMs are defined in Chapter 2 of the Guideline.

When a design equivalent material class (DEMAC) is assigned to a material, it implies that the material exhibits in situ shear strength, stiffness and flexibility properties similar to those of a newly constructed material of the same class. For example, a layer in an existing pavement structure classified as a G2 design equivalent would indicate that the material is considered to be equivalent to a G2 for design purposes, based on the available test evidence. For brevity, a DEMAC will be denoted DE-G2, for example.

The materials classification system described in the next section provides a consistent method to evaluate and document the necessary evidence to support the material classification.

A.3. MATERIAL CLASSIFICATION SYSTEM

A.3.1. Theory of Holistic Approach

The material classification system provides a framework for the rational synthesis of several different test indicators. The outcome of the assessment becomes more reliable as more test indicators are added to the assessment. This is because each test typically explains only a small part of the cohesive or frictional elements of material behaviour. More complex tests, like triaxial tests, may evaluate these two elements together, but will do so only for a specific moisture or bitumen content. The use of other indicators will still be needed to determine how the material will behave if the moisture state or bitumen content changes. Since each test provides only a partial explanation of the material's behaviour, the reliability of the assessment can be greatly increased by increasing the sample size, and by adding more indicators (i.e. test types) to the assessment. The system is therefore a holistic assessment, which works best when a comprehensive range of test indicators are used.

The theory underlying the method is based on Fuzzy Logic and Certainty Theory. The development of the method is described in detail by Jooste *et al* (2007) and the validation of the method is described by Long (2009). A summary of the theoretical process to classify a material is as follows:

1. If H is the hypothesis to be tested, then the certainty that the hypothesis is true is designated as C(H), which has a value of 1.0 if H is known to be true, 0.0 if H is unknown and -1.0 if H is known to be false. In the context of the present study, H could for example be the hypothesis that the base layer is a DE-G1.

2. The value of C(H) is determined by applying rules which are based on experience or domain knowledge. Each rule has a certainty factor (CF) associated with it, to reflect the level of certainty in the available evidence, or in the knowledge on which the rule is based.
A typical rule may be:
If [PI < 4] then [Material is a DE-G1] With Certainty CF

3. The certainty factor of a rule, CF, is modified to reflect the level of certainty in the evidence. This gives the modified certainty factor CF′, calculated simply as:

$$CF' = CF \times C(E) \qquad (A.1)$$

Where C(E) is a number between 0 and 1, indicating that the evidence in support of the hypothesis is either completely absent (C(E) = 0.0) or known to be present with absolute certainty (C(E) = 1.0).
4. To get C(H|E), which is the updated certainty that the hypothesis H is true, given the evidence E, the following composite function is applied:

$$\text{If } C(H) \geq 0 \text{ and } CF' \geq 0 \text{ then: } C(H|E) = C(H) + [CF' \times (1-C(H))] \qquad (A.2)$$

$$\text{If } C(H) \leq 0 \text{ and } CF' \leq 0 \text{ then: } C(H|E) = C(H) + [CF' \times (1+C(H))] \qquad (A.3)$$

If C(H) and CF′ have opposite signs, then:

$$C(H\,|\,E) = \frac{C(H)\,CF'}{1-\min(|C(H)|,|CF'|)} \qquad (A.4)$$

In the application of the above methodology for material classification, the certainty factor CF associated with a specific test is assigned based on domain knowledge and experience. If the test is known to be a good overall indicator of cohesion, frictional resistance or both, then CF will tend to be higher. CF can also be adjusted based on the sample size and range of sampled values. For small sample sizes, CF can be lowered to reflect decreased confidence in the available evidence.

The steps and equations outlined above provide a general method for consistently evaluating the certainty that a hypothesis is true, given uncertain and vague rules and evidence. A generalized and simplified example of the method's application for materials classification is outlined below:

1. We want to test the hypothesis H that the material for which we have information is a graded natural gravel (DE-G4). To do this, we formulate the following rules:

If [Material is Natural Gravel] and [PI < 4] then [Material is a DE-G4] with CF = 0.4
If [Grading conforms to G4 Envelope] then [Material is a DE-G4] with CF = 0.3

2. We now obtain samples and measure the PI and grading. The certainty factors can be adjusted based on the sample size.
3. We start with the first available evidence (PI test). At this stage C(H) = 0. Since CF = 0.4 for the first rule concerning PI, we use Equation A1 and A2 to calculate the updated certainty for the hypothesis that the material is a DE-G4 (C(H|E)).
4. The updated certainty C(H|E) becomes the new starting certainty C(H) for the second rule which interprets the grading. We again apply Equation A.2 to calculate the new value for C(H|E).
This process can be applied for each material class to obtain a relative indication of how much the available test data point to each class.
The following sections give more details on the process.

A.3.2. Step by Step Material Classification

The Certainty Theory approach involves an assessment of how well the available evidence suits a given hypothesis. In the present context, the evidence would be available test data, and the hypothesis to evaluate would be that the material conforms to a specific material class.
The method involves the following steps:

Step 1: For each of the available material tests, determine and report the 90th percentile, median and 10th percentile values from the available observations. For those tests for which a rating system is provided, use the ratings at each observation to determine the required statistics. Where there is only one observation available, simply report the observation as the median value.

Step 2: Determine the certainty factor associated with each of the available tests (i.e. CF as defined in Section A.4). This certainty reflects the confidence that we have in each test to provide an accurate indication of the in situ shear strength and stiffness of the material. Details related to this step are provided in Section A.3.3.

Step 3: Adjust the relative certainty determined in Step 2 to take account of sample size. This adjustment decreases the confidence for smaller sample sizes. Details related to this step are provided in Section A.3.4.

Step 4: Select a likely material class (e.g. DE-G4) for the layer in question.

Step 5: For each of the available tests, determine the expected range of values for each DEMAC for the selected material from Table A.3 (Granular) or Table A.11 (Cemented). For example, if the material in question is a DE-G4, and the test is the soaked CBR at 98% Mod. AASHTO density, we will use Table A.3 to obtain the expected range of CBR values for a G4 (i.e. 80 to 99%). For tests that involve a rating system, as defined in Section A.4, the rating values corresponding to different material classes are shown in Table A.3 (Granular) and Table A.11 (Cemented).
Some tests or indicators have expected ranges for the material classes for different material types, compaction levels or specimen diameter. For example, in Table A.3, the Plasticity Index has different values for crushed stone, natural gravel, gravel selected per test or indicator.

Step 6: For each test, determine how much the 10th percentile to 90th percentile range overlaps with the expected range of values for the material. This provides the relative certainty that the test data points to the material class in question (i.e. factor C(E) as defined in Section A.3.1). Details of how to perform this calculation are provided in Section A.3.5.

Step 7: For each test, use the certainty factor CF from steps 2 and 3 and the certainty of evidence C(E) from Step 6, to update the certainty that the material tested conforms to the class selected in Step 4. This calculation then provides the relative certainty
that the material belongs to the selected DEMAC, given the available evidence (i.e. C(H|E) as defined in Section A.3.1). Details on these calculations are provided in Section A.3.6.

Step 8: Repeat Steps 4 to 7 for each likely material class. For example, if we are performing a classification for an unbound granular base, we may evaluate the certainty associated with classes DE-G1 to DE-G5.

Step 9: Select the material with the highest certainty given the available evidence. This material class is assigned to the layer in question.
Properly document the evidence and calculations.

A.3.3. Certainty Factors for Different Tests and Indicators

Because most pavement materials tests provide only a partial indication of the shear strength and stiffness of a material, a certainty factor is assigned to each test indicator. This certainty factor represents the factor CF as defined in Section A.3.1. In essence, CF represents the subjective confidence in the ability of a test to serve as an accurate indicator for material strength and stiffness. The value of CF can range from 0 to 1, with a value of 1 indicating absolute confidence in a test or indicator (a highly unlikely assignment).

Suggested certainty factors for the tests and indicators used in the classification system are provided in Table A.3 (granular materials) and Table A.11 (cemented materials) (Table 8 for BSMs is not included here but in TG2 Manual, ref.15). The ratings shown in these tables are based on a subjective assessment of the completeness and appropriateness of each test or indicator. Engineers can adjust these values to take account of experience or specific project situations, but the assumed values should be reported to clients. If the assumed values deviate substantially from those suggested in the tables, the assumed values must be motivated in the assessment report.

A.3.4. Adjustment for Sample Size

Small sample sizes, i.e. one or two observations are not uncommon in pavement condition assessments. However, this affects the certainty with which a material class is assessed. To take account of this, the Certainty Factor (CF) associated with each test is adjusted to take account of the sample size. Table A.1 shows the recommended adjustment factors based on sample size. These factors are applied by multiplying the factor from Table A.1 with the CF factor for the test from Table A.3 (Granular) and Table A11 (Cemented) (Table A.8 (BSM) not included).

Table A1: Recommended Adjustment of CF based on Sample size.

SAMPLE SIZE (No of observations)	ADJUSTMENT FACTOR
1	0,2
2	0,3
3	0,6
4 to 6	0,7
6 or greater	1,0

A.3.5. Assessing the Relative Certainty of Evidence

The material classification method assesses the certainty that a material can be classified as a particular material class. This assessment is vague and uncertain because of the incompleteness of most tests and because a sampling estimate is used. The incompleteness is taken into account with the Certainty Factor (Section A.3.3), but the variation in the tests results needs to be considered.

The method for achieving this is illustrated in Figure A.3. The figure shows the CBR limits associated with material classes DE-G5, DE-G6 and DE-G7. Also shown is a triangle which is determined as follows: left bottom corner is the 10th percentile value, top corner is the median value and right bottom corner is the 90th percentile value.

The triangle represents the available evidence in a relative manner. The height of the triangle is given a fixed value of 1.0. The total area of the triangle and the portion of the triangle that falls within the DE-G6 class are calculated. The relative area that overlaps with the DE-G6 class gives us a relative indication of how strongly the CBR evidence points to a DE-G6 class. In the context of the certainty theory methodology, we assume that the relative area that overlaps with the material class in question, gives us the factor C(E) as defined in Section A.3.1.

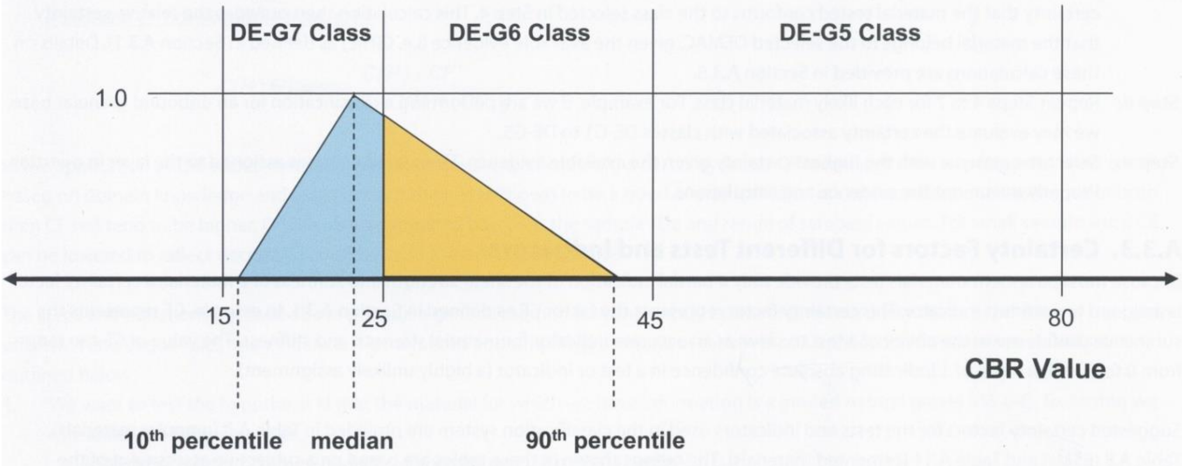

Figure A.3 Determining Relative Conformance of Evidence to Material Class Limits

A.3.6. Updating Material Classification for Available Evidence

The objective of the assessment is to determine the certainty associated with the hypothesis that a material conforms to a selected DEMAC. For example, if the material selected for evaluation is a DE-G6, and we want to obtain the relative certainty that the material is indeed a DE-G6. As defined in Section A.3.1, the certainty for this hypothesis is C(H), which is initially zero, but which will increase when we consider tests for which the results conform partly to the range expected for a DE-G6 material.
The certainty factors for the different tests, combined with the adjustment for sample size, provide the certainty factor CF associated with each test (Section A.3.3). The comparison of the test results with the expected limits for the DEMAC in question (as shown in Figure A.3 and discussed in Section A.3.5) provides us with the certainty that evidence is present, C(E). These are all the factors needed to calculate an updated certainty for the hypothesis that the material tested conforms to the selected DEMAC, i.e. C(H|E) as defined by Equations A.1 to A.4 (Section A.3.1).

Usually, the calculation of C(H|E) mostly involves repeated application of Equation A.2. Initially, C(H) is zero. Then, CF′ is calculated using Equation A.1, and then C(H|E) using Equation A.2. Then, the next test type is evaluated, which has a new CF and C(E) associated with it. The CF′ is then recalculated. For the new test type, the certainty C(H) is set equal to C(H|E) determined from the previous test type. The C(H) and CF′ in Equation A.1 are used to calculate the new C(H|E). This process is repeated for each test type to obtain an overall certainty that the material conforms to the selected DEMAC.

Once the overall certainty that the material conforms to the selected DEMAC is assessed, the next likely class is selected and the process repeated using the same set of information. In some instances, this evaluation may require that the conformance to five or more classes be evaluated. Although this seems cumbersome, the calculations are simple and the process can easily be automated using a spreadsheet macro or a computer program.

A software program to do the material classification is available on www.asphaltacademy.co.za/bitstab. The software runs on the website and it is not necessary to download the software to a local computer. A Microsoft Excel template for preparing and uploading the data can be downloaded from the website.

A.4. TESTS AND INTERPRETATION OF RESULTS

This section details the tests that are used for the material classification, the interpretation of the test results and the certainty factors. Three materials are covered: untreated granular materials, bitumen stabilised materials and cement stabilised materials.

The material classification method is relatively new, and although it has been well validated, especially for granular and cemented materials (Long, 2009), it is possible that further refinements may be necessary. If such refinements are made, the most up to date limits and certainty factors will be posted on www.asphaltacademy.co.za/bitstab. It is therefore recommended that before commencing the material classification process, the website is checked for any changes in values or tests.

A.4.1. Granular Materials

The classification of granular materials is aligned with TRH14 (1985). The indicators and tests for the classification of unbound granular materials are detailed in Table A.2, and the relevance of the test or indicator is explained. The interpretation of the test results are given in Table A.3. The values shown have been validated and provide consistent, reasonable results (Long, 2009). The interpretation of consistency, visible moisture, grading and historical performance requires the determination of a rating. Details on the ratings are given in Table A.4, with additional information in Table A.5 (historical performance), Table A.6 (consistency) and Figure A.4 (grading).

Table A.2: Indicators and tests for Classification of Unbound Granular Materials.

Test or Indicator	Relevance for Material Classification	Interpretation or rating.	Comments
Soaked CBR	When soaked, tests mainly the frictional strength component of shear strength.	Table A.3	Test relevance and interpretation is based on TRH14 specifications.
Percent passing 0.075 mm Sieve (Fines)	Impacts on the density that can be achieved, and on the bearing strength of the material. As such, relates mainly to frictional component of shear strength.	Table A.3	Ideal range is 6 to 10%. At less than 4% fines, density is difficult to achieve. Shear strength reduces when fines exceed roughly 13% (Hefer and Scullion, 2002; Gray, 1962).
Relative Density	Relates to the density of packing of particles, and hence to the potential to develop frictional resistance.	Table A.3	Test relevance and interpretation is based on TRH14 specifications.
DCP Penetration	Indicator for overall shear strength. Sensitive to density, moisture content, particle strength, grading and plasticity.	Table A.3	Test relevance and interpretation is based on experience and ranges published Kleyn (1984).
FWD back-calculated Stiffness	Provides a direct but relative indication of the stiffness under dynamic loading for most materials. Likely to be highly correlated to shear strength at small strains.	Table A.3	Test relevance and interpretation ranges based on experience in southern Africa.
Consistency Rating	Provides a rough indication of material density and stiffness.	Table A.4	Rating based on material consistency evaluation from test pits.
Plasticity Index	Determines the influence of water on shear strength. For a fixed maximum aggregate size, shear strength is greatly reduced with an increase in PI.	Table A.3	Based on TRH14. Test relevance and main effects related to shear strength are reported in Hefer and Scullion (2002); and in Gray (1962).
Visible and Measured moisture content	The relative moisture content is the measured moisture content, relative to the optimum moisture content for the material. It provides an indication of the degree of saturation and the relative cohesive strength.	Table A.3 and Table A.4	Rating and limits are based on experience, and on specifications reported by Hefer and Scullion (2002). These include the specifications of New South Wales (1997), and Queensland (1999).
Grading Assessment Rating	Rating quantifies the conformance of the material grading to applicable specifications. Good conformance to grading indicates increased frictional resistance.	Table A.4 And Figure A.4	Rating requires that the relative conformance to the appropriate grading be quantified. This value is then used to obtain an overall rating for grading based on material type.
Grading Modulus	Quantifies the relative amount of fines in the material. As such, it influences the ability of the material to develop interlock between coarse particles.	Table A.3	Based on TRH14 and on COLTO (1998) specifications.
Historical Performance	The historical performance for the base and subgrade can be isolated with some confidence using past traffic and observed condition.	Table A.5	Based on experience and existing guidelines (e.g. TRH12, 1998).

Table A.3 Interpretation of Indicators and Tests for Classification of Unbound Granular Materials

Test or Indicator	Material	Design Equivalent Material Class										CF
		DE-G1	DE-G2	DE-G3	DE-G4	DE-G5	DE-G6	DE-G7	DE-G8	DE-G9	DE-G10	
Soaked CBR (%)	CS (98%)	> 100	80 to 99			45 to 79	25 to 44	15 to 24	10 to 14	7 to 9	< 7	0.4
	NG (95%)					> 45	25 to 44	15 to 24	10 to 14	7 to 9	< 7	
	NG/GS (93%)						> 25	15 to 24	10 to 14	7 to 9	< 7	
	SSSC (90%)							> 15	10 to 14	7 to 9	< 7	
P0.075 (%)	CS	4 to 12										0.3
	NG				5 to 15	13 to 20	15 to 25	25 to 30	30 to 40	40 to 50	> 50	
	GS					5 to 15	13 to 20	15 to 25	25 to 30	30 to 40	> 40	
	SSSC						0 to 10	10 to 15	15 to 20	20 to 30	> 30	
Relative compaction density	All	> 1.02	1.00 to 1.0~~2~~ 1	0.98 to ~~1.0~~ 0.99		0.95 to 0.9~~8~~ 7	0.93 to 0.9~~5~~ 4		< 0.93			0.3
DCP Pen (mm/blow)	All	< 1.40	1.40 to 1.79	1.80 to 1.99	2.00 to 3.69	3.70 to 5.69	5.7 to 9.09	9.1 to 13.99	14 to 18.99	19.0 to 25.0	> 25	0.4
FWD Backcalc. Stiffness (MPa)	All	> 600	500 to 600	400 to 499	300 to 399	200 to 299	150 to 199	100 to 149	70 to 99	50 to 69	0 to 49	0.3
Plasticity Index	CS	< 4	4 to 5	5 to 7	7 to 10	> 10						0.4
	NG			< 5	5 to 6	6 to 10	10 to 12	> 12				
	GS						< 11	11 to 12	12 to 15	> 15		
	SSSC							< 12	12 to 14	14 to 20	> 20	
Relative moisture (%)	CS	< 60	60 to 65	65 to 80	80 to 90	90 to 100	> 100					0.3
	NG			< 65	65 to 70	70 to 80	80 to 100	> 100				
	GS						< 80	80 to 90	90 to 100	> 100		
	SSSC							< 90	90 to 100	100 to 120	> 120	
Grading modulus	NG				2.0 to 2.6	1.5 to 2.6	1.2 to 2.7	< 1.2				0.2
	GS						1.2 to 2.5	0.75 to 2.7	0.75 to 2.7	0.75 to 2.7	< 0.75	
Rating	All	0.5 to 1.5	1.5 to 2.5	2.5 to 3.5	3.5 to 4.5	4.5 to 5.5	5.5 to 6.5	6.5 to 7.5	7.5 to 8.5	8.5 to 9.5	9.5 to 10.5	N/A

updated.

Abbreviations: CS = crushed stone, NG = natural gravel, GS = gravel soil, SSSC = sand, silty sand, silt, clay; 98%, 95%, 93%, 90% are Mod. AASHTO densities.

Table A.4 Rating of Indicators and Tests for Classification of Unbound Granular Materials

Test or Indicator	Material	Rating 1	2	3	4	5	6	7	8	9	10	CF
Consistency (see Table A.6)	CS	Very dense	Dense	Medium dense	Loose	Very loose						0.2
	NG			Very dense	Dense	Medium dense	Loose	Very loose				
	NG/GS					Very dense	Dense	Medium dense	Loose	Very loose		
	SSSC						Very stiff	Stiff	Firm	Soft	Very soft	
Visible moisture	CS	Dry	Slightly moist	Moist	Very moist	Wet						0.2
	NG		Dry	Slightly moist	Moist	Very moist	Wet					
	GS					Dry	Slightly moist	Moist	Very moist	Wet		
	SSSC						Dry	Slightly moist	Moist	Very moist	Wet	
Grading (see Figure A.4)	CS	1	2	3	4							0.4
	NG				1	2	3	4				
	GS					1	2	3	4			

Abbreviations: CS = crushed stone, NG = natural gravel, GS = gravel soil, SSSC = sand, silty sand, silt, clay

Table A.5 Rating of Historical Performance

Layer	Condition Description	Traffic Accommodated to Date (MESA)				
		< 0.5	0.5 to 1	1 to 3	3 to 10	> 10
Base[1]	No visible rutting, deformation, pumping or potholes, surfacing mostly intact. Minor patching only.	Difficult to assess		2	1	1
	Less than 8 mm narrow rutting in wheelpath, minor pumping and traffic-related cracking. Minor patching.	Difficult to assess	4	3	2	1
	8 to 12 mm narrow rutting in wheelpath, some deformation, shoving and/or pumping. Frequent patching noted.	7	5	4	3	Difficult to assess
	More than 12 mm narrow rutting in wheelpath, severe and frequent shoving, pumping and/or deformation. Frequent patching.	9	7	5	Difficult to assess	
Subbase[2]	No wide, subgrade relative rutting visible.	Difficult to assess		7	6	5
	Suspect some subgrade deformation occurred, as shown by wide, subgrade related rutting (<10 mm depth), and slight undulation and/or subgrade related failures.	Difficult to assess		8	7	6
	Strong evidence of subgrade related rutting (>10 mm depth) and/or definite signs of subgrade related failures.	10	9	8	Difficult to assess	

Notes:

1. Assessment is only valid if there are no surfacing related problems (e.g. stripping, brittleness, rutting) which may have caused a rapid deterioration in the base layer. Also, assessment is not valid if overlay or surface seal was recently placed.
2. Assessment is only valid if an overlay or surface seal was no recently placed.

Table A.6 Guidelines for Consistency

Material Type	Consistency	Description of Layer Condition
Coarse granular materials	Very Loose	Very easily excavated with spade. Crumbles very easily when scraped with geological pick.
	Loose	Small resistance to penetration by sharp end of geological pick.
	Medium Dense	Considerable resistance to penetration by sharp end of geological pick.
	Dense	Very high resistance to penetration of sharp end; and requires blows of geological pick for excavation.
	Very Dense	Very high resistance to repeated blows of geological pick; and requires power tools for excavation.
Cohesive soils	Very Soft	Geological pick head can easily be pushed in to the shaft of handle; easily moulded by fingers.
	Soft	Easily penetrated by thumb; sharp end of geological pick can be pushed in 30 to 40 mm; moulded with some pressure.
	Firm	Indented by thumb with effort; sharp end of geological pick can be pushed in up to 10 mm; very difficult to mould with fingers; can just be penetrated with an ordinary hand spade.
	Stiff	Penetrated by thumb nail; slight indentation produced by pushing geological pick point into soil; cannot be moulded by fingers; requires hand pick for excavation.
	Very Stiff	Indented by thumb nail with difficulty; slight indentation produced by blow of geological pick point; requires power tools for excavation.

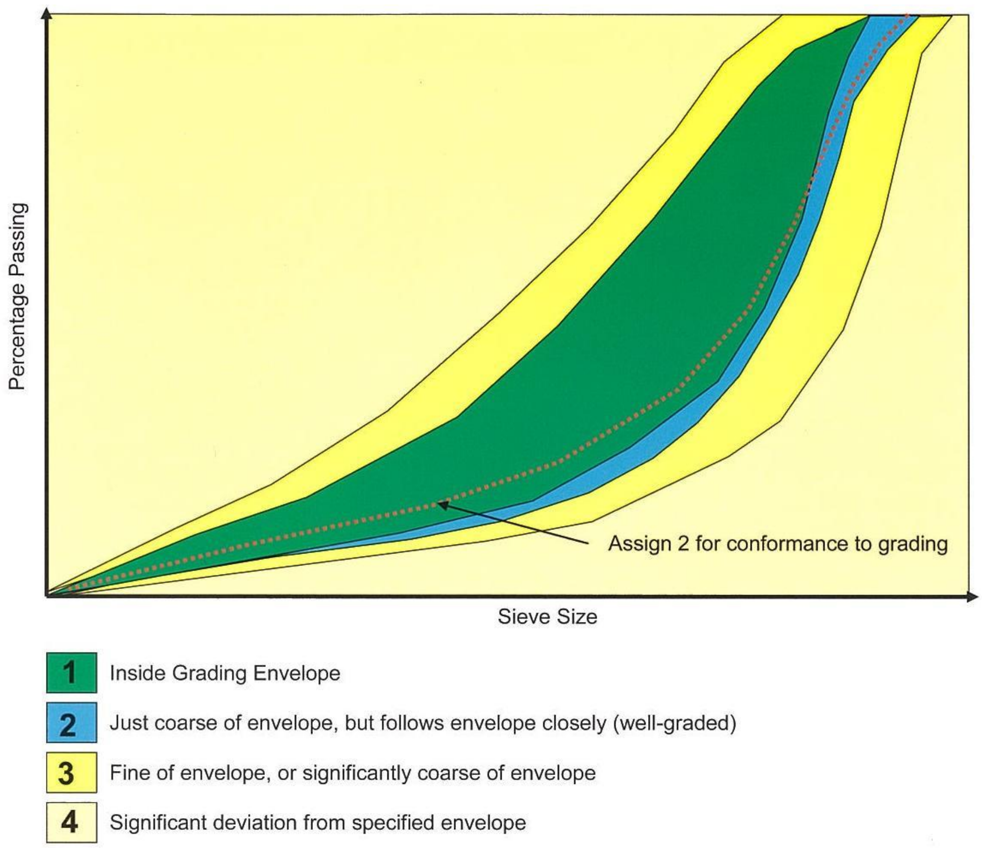

Figure A.4 Interpretation of Grading to Quantify Relative Conformance to Grading (Granular)

A4.2 Bitumen Stabilised Materials

Bitumen stabilised materials are not dealt with here as it is not often used but the determination of the DEMAC for bitumen stabilised materials are adequately dealt with in the TG2 manual.

A.4.3. Cement Stabilised Materials

The classification of cement stabilised materials focuses on the degree of cementation still present. The material is classified as a rating, from 1 to 3. This rating scheme has the following relationship between the rating and the material classes as defined in TRH14 (1985).

Rating 1: Indicates condition similar to recently constructed C1, C2, or C3 material.
Rating 2: Indicates condition similar to recently constructed C4 material.
Rating 3: Indicates material is either ineffectively stabilised, or has deteriorated to an equivalent granular state. These materials should be regarded as unbound granular materials and the classification guidelines in Section A.4.1 should be applied.

The indicators and tests for the classification of cemented materials are detailed in Table A.10, and the relevance of the test or indicator is explained. The interpretation of the test results are given in Table A.11. The values shown have been validated and provide consistent, reasonable results (Long, 2009).

Table A10: Indicators and tests for classification of Cement Stabilised Materials.

Test or indicator	Relevance for material Class.	Rating	Comments
DCP Penetration	Indicator for overall shear strength. Sensitive to density, moisture content, particle strength, grading and plasticity.	Table A.11	Test relevance and interpretation is based on experience and ranges published Kleyn (1984).
FWD Backcalculated Stiffness	Provides a direct but relative indication of the stiffness under dynamic loading. Likely to be highly correlated to shear strength at small strains for most materials.	Table A.11	Test relevance and interpretation ranges based on experience in southern Africa.
Consistency Rating	Provides a rough indication of the degree of cementation of the material.	Table A.11	Rating based on material consistency evaluation from test pits and on the SANRAL M1 Manual (SANRAL, 2004).
Evidence of Active Cement	Quantifies the confidence that material is acting as a cohesive, cement stabilised layer.	Table A.11	None

Table A.11 Interpretation of Indicators and Tests for Classification of Cemented Materials from Field observations

Rating	1	2	3	
Test or Indicator	Indicates condition similar to recently constructed C1, C2 or C3 Material	Indicates condition similar to recently constructed C4 material.	Indicates material is either ineffectively stabilised or deterio-ration to an equivalent granular state.	CF
Consistency	Hand-held specimen can be broken with hammer head with single firm blow. Similar appearance to concrete.	Material crumbles under firm blows of sharp geological pick point. Grains can be dislodged with some difficulty under a knife blade.	Some material can be crumbled by strong pressure between fingers and thumb. Disintegrates under a knife blade to a friable state.	0.2
	Firm blows of sharp geological pick point.	Cannot be crumbled between strong fingers. Some material can be crumbled by strong pressure between thumb and hard surface. Disintegrates under light blows of a hammer head to a friable state.		
DCP Penetration (mm/blow)	< 1.50	1.5 to 3.0	> 3	0.4
FWD Backcalculated Stiffness (MPa)	> 1 200	500 to 1 200	< 500	0.3
Evidence of Active Cement	Clearly visible in material colour and consistency. Clear indication of active cement, based on chemical tests.	No cementation visible, slight indication of active cement, based on chemical tests.	No indication of active cement, either in material colour and consistency or from chemical tests.	0.3

A.5. CONFIDENCE ASSOCIATED WITH ASSESSMENT

The confidence in the certainty associated with the material classes depends on the number of tests or indicators used and the certainty factors associated with the tests and indicators. The strength of confidence in our assessment is thus quantified by the certainty of the assessment, and this is an indirect indicator of the reliability of any design which is based on this assessment. Table A.12 provides some guidelines to assess the confidence associated with the material classification.

Table A.12 Relative Confidence of Materials Classification

Final value of C(H|E)	**Confidence in classification**
< 0.3	**Very low confidence.** It is strongly recommended that more data be gathered to enable a more confident assessment to be made.
0.3 to 0.5	**Low confidence.** Suitable only for situations where the existing pavement condition and age is such that structural rehabilitation will not be considered or is very unlikely.
0.5 to 0.7	**Medium**. Suitable for situations where the existing pavement condition and age is such that structural rehabilitation is unlikely, or for which the condition and/or other factors predetermines the treatment type.
>0.7	**High.** This is the minimum recommended certainty for situations where structural rehabilitation is likely, and for which the rehabilitation design will rely completely on the quality and state of

	existing pavement layers.

Materials codes and abbreviated specifications, treated materials.

A.6. WORKED EXAMPLE

The following paragraphs illustrate the application of the method described in Section A.3. This example uses data from an actual pavement rehabilitation investigation, but with some slight adjustments to clearly illustrate the concepts of the method. The example involves an assessment of an upper subbase layer for the eastbound lane of a planned rehabilitation project 18 km long.

Based on the condition of the road, the construction history and the deflection patterns, the road was designated as a single uniform design section. All available results are therefore assessed together. The available information consists of the following:
Materials test data from nine test pits. Available test data include: material description, relative density, moisture content, DCP penetration, grading analyses, CBR and PI.
173 FWD deflections with backcalculated stiffnesses for all layers.
Table A.13 summarizes some of the test indicators. The grading analyses are summarized in Figure A.6. In the test pits, the material was described as a dense weathered dolerite natural gravel in all instances and therefore the classification system for granular materials is appropriate.

Table A.13 Example Materials Test Data

Station (Km)	Relative Density	CBR (%)	% Passing 0.075 mm Sieve	Moisture as % of Optimum	GM	PI	Consistency Rating[1]	Grading Rating[2]	DCP Pen (mm/blow)[3]
1.5	1.03	70	11	70	2.34	9	4	4	1.8
2.7	0.87	24	4	108	2.7	10	4	6	4.8
4.3	0.94	64	5	91	2.67	9	4	5	-1
4.9	1	66	6	96	2.65	7	4	5	-1
7.9	1	70	13	67	2.17	8	4	4	-1
9.2	1	100	3	75	2.68	8	4	5	-1
12.5	1	90	12	63	2.24	5	4	4	2.4
14.3	0.98	80	6	72	2.59	8	4	4	1.4
17.5	0.94	N/R	10	48	2.24	6	4	5	-1
10th Percentile	**0.93**	**52**	3.8	**60**	**2.2**	**6**	4	4	**-1.0**
Median	**1.00**	**70**	6.0	**72**	**2.6**	**8**	4	5	**-1.0**
90th Percentile	**1.01**	**93**	12.2	**98**	**2.7**	**9**	4	5	**2.9**
Observations	**9**	**8**	9	**9**	**9**	**9**	9	9	**9**

Note:
1. Consistency rating determined from Table A.4 and Table A.6.
2. Grading rating determined from Table A.4 and Figure A.4.
3. For DCP penetration, a value of -1 indicates refusal.

The backcalculated stiffnesses for the subbase were as follows:
10th Percentile = 189 MPa
Median = 466 MPa
90th Percentile = 581 MPa
For most of the available tests, the results can be directly evaluated by means of the interpretation guidelines provided in Section A.4.

However, for the consistency and grading, the test results first have to be converted to a rating, to facilitate a numerical evaluation of results. The ratings assigned for these indicators are summarized in Table A.13.
Once all the tests have been quantified, we can summarize the available tests, their certainty factors and their sample statistics. For this example, the certainty factors from Table A.3 were adopted. Since the sample size exceeds six for all tests, the adjustment factor for sample size (from Table A.1) is 1.0 in all cases. The available test data and certainty factors are summarized in Table A.14.

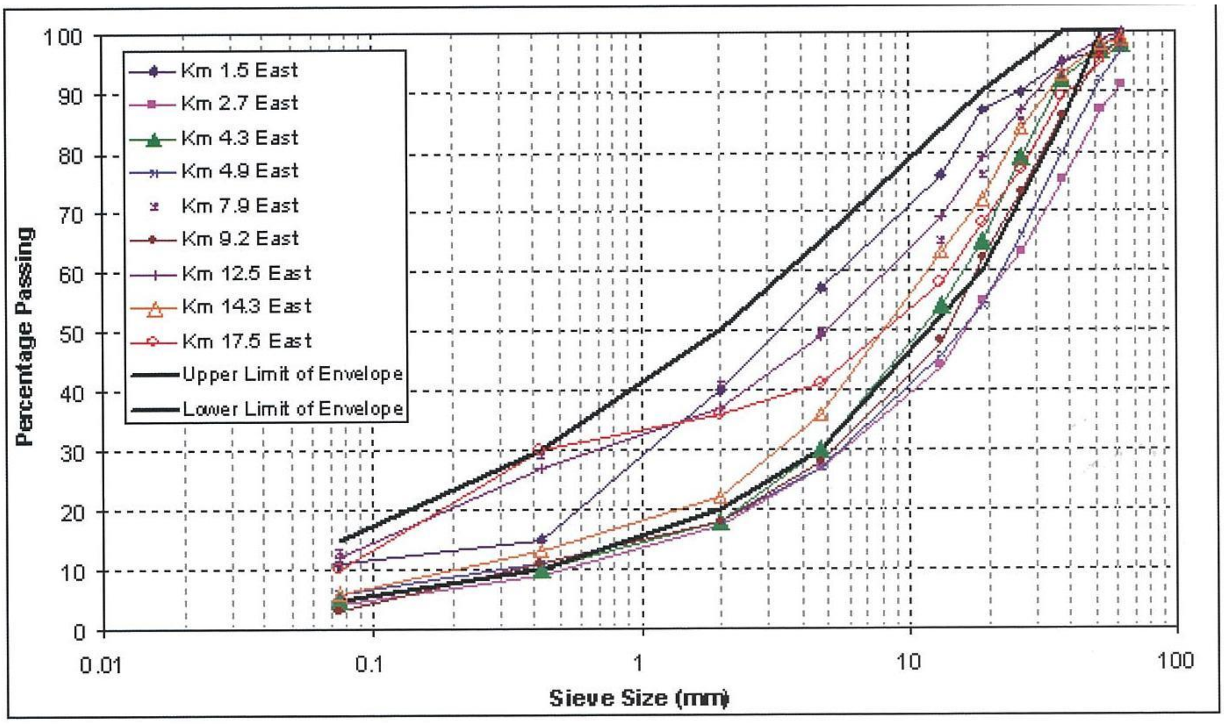

Figure A.6 Grading Analyses for Worked Example

In Table A.14, CF is the certainty factor related to the test type, and CF' is simply CF adjusted to take account of sample size. In this case, CF' is equal to CF because the sample size is greater than 6 in all cases. Columns 6, 7 and 8 represent the relative certainty that the test evidence points to a DE-G4, DE-G5 or DE-G6 design equivalent material class.

The factors C(E) are determined using the method described in Section A.3.1. Figure A.7 shows an example of the detailed calculation of C(E) for FWD Backcalculated Stiffness. This calculation relies on the FWD stiffness limits recommended in Table A.3 and on the sample statistics shown highlighted for FWD stiffness in Table A.14.

Table A.14 Worked Example, Summary of Test Data and Certainty Factors

Test	CF	CF'	10th %	Median	90th %	C(E) DE-G4	C(E) DE-G5	C(E) DE-G6
DCP Penetration	0.4	0.4	-1.0	-1.0	2.9	0.05	0.00	0.00
CBR (NG)	0.4	0.4	52	70	93	0.18	0.79	0.0
P0.075 (NG)	0.3	0.3	3.8	6.0	12.2	0.92	0.0	0.0
Relative Density	0.3	0.3	0.93	1.00	1.01	0.20	0.21	0.02
FWD Stiffnesses	0.3	0.3	189	466	581	0.30	0.11	0.00
Consistency Rating	0.2	0.2	4	4	4	1.0	0.0	0.0
PI (NG)	0.4	0.4	6	8	9	0.0	1.0	0.0
Rel. Moisture (NG)	0.3	0.3	60	72	98	0.33	0.54	0.08
Grading Rating	0.4	0.4	4	5	5	0.25	0.75	0.0
GM (NG)	0.2	0.2	2.2	2.6	2.7	0.44	0.50	0.06
Column	1	2	3	4	5	6	7	8

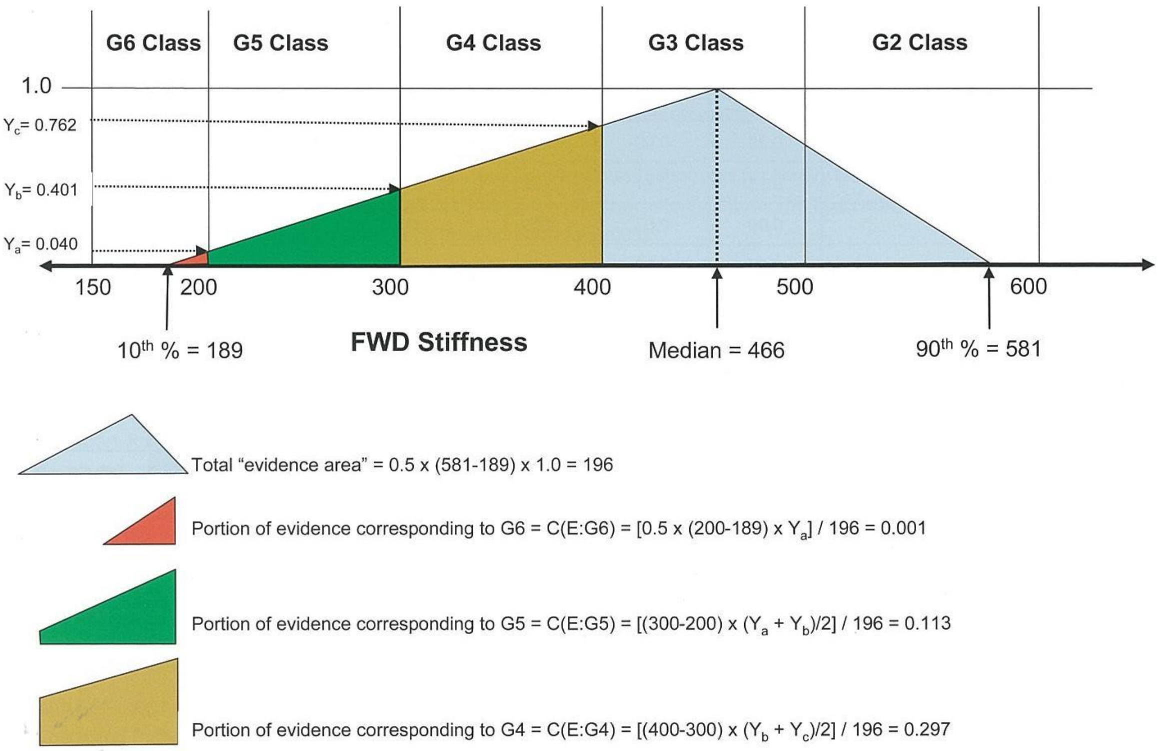

Figure A.7 Example of C(E) Calculations for FWD Backcalculated Stiffness Sample

Table A.15 shows the final adjusted certainty factors (CF') for a DE-G4, DE-G5 and DE-G6 material, and also the cumulative certainty that the material is a DE-G4, DE-G5 or DE-G6 (i.e. C(H|E)). The final cumulative certainty for these three material classes is shown in the bottom row. The classification method shows that most of the evidence points to the material being a DE-G5, and some evidence also points to a DE-G4. In comparison to a DE-G4 and DE-G5, there is comparatively little information to suggest that the material is a DE-G6.

Table A.15 Worked Example, Summary of Certainty Associated with DE-G4, DE-G5 and CE-G6

Test	CF'			C(H-DEGX|E)		
	DE-G4	DE-G5	DE-G6	DE-G4	DE-G5	DE-G6
DCP Penetration	0.02	0.00	0.00	0.02	0.00	0.00
CBR	0.00	0.40	0.00	0.02	0.04	0.00
P0.075	0.28	0.00	0.00	0.29	0.4	0.00
Relative Density	0.13	0.11	0.02	0.38	0.47	0.02
FWD Stiffnesses	0.09	0.03	0.00	0.44	0.49	0.02
Consistency Rating	0.20	0.00	0.00	0.55	0.49	0.02
PI	0.00	0.40	0.00	0.55	0.69	0.02
Measured Moisture	0.06	0.13	0.09	0.58	0.73	0.11
Grading Rating	0.10	0.30	0.00	0.62	0.81	0.11
GM	0.16	0.09	0.20	0.68	0.83	0.29
Final Assessment of Relative Certainty for				**DE-G4 = 0.68**	**DE-G5 = 0.83**	**DE-G6 = 0.29**
Most likely Materials Class is a G5 Design Equivalent Class **Relative Certainty associated with this outcome = 0.83** Confidence associated with this outcome is **High**. Assessment is suitable for situations where structural rehabilitation is required, or for which the rehabilitation design will rely completely on the state of existing layers.						

Note: CF' calculated with Equation A.1
C(H-G4/G5/G6|E) calculated with Equation A.2, A.3 or A.4

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Materials Symbols and abbreviated specifications: TRH4/TRH14.

Material symbols and abbreviated specifications used in the Catalogue designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense - graded unweathered crushed stone; Maximum size 37,5 mm; 86 - 88 % apparent relative density; Soil fines PI < 4
	G2	Graded crushed stone	Dense - graded crushed stone; Maximum size 37,5 mm;100 - 102 % Mod. AASHTO or 85 % bulk relative density; Soil fines PI < 6
	G3	Graded crushed stone	Dense - graded stone and soil binder; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO ; Soil fines PI < 6
	G4	Crushed or natural gravel	Minimum CBR = 80 % @ 98 % Mod. AASHTO; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO; PI < 6; Maximum Swell 0,2 % @ 100 % Mod. AASHTO. For calcrete PI ≤ 8
	G5	Natural gravel	Minimum CBR = 45 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 10; Maximum swell 0,5 % @ 100 % Mod. AASHTO *
	G6	Natural gravel	Minimum CBR = 25 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12; Maximum swell 1,0 % @ 100 % Mod. AASHTO *
	G7	Gravel / Soil	Minimum CBR = 15 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G8	Gravel / Soil	Minimum CBR = 10 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G9	Gravel / Soil	Minimum CBR = 7 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G10	Gravel / Soil	Minimum CBR = 3 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; or 90% Mod. AASHTO

* For calcrete PI ≤ 15 on condition that the Linear Shrinkage (LS) does not exceed 6 %.

** GM = Grading Modulus (TRH14, 1985) $= \dfrac{300 - [P_{2,00mm} + P_{0,425mm} + P_{0,075mm}]}{100}$
where $p_{2,00}$ etc., denote the percentage passing through the sieve size.

*** For calcrete PI ≤ 17 on condition that the Linear Shrinkage (LS) does not exceed 7 %.

Materials and symbols used for Stabilised materials

Material symbols and abbreviated specifications used in the Catalogue designs

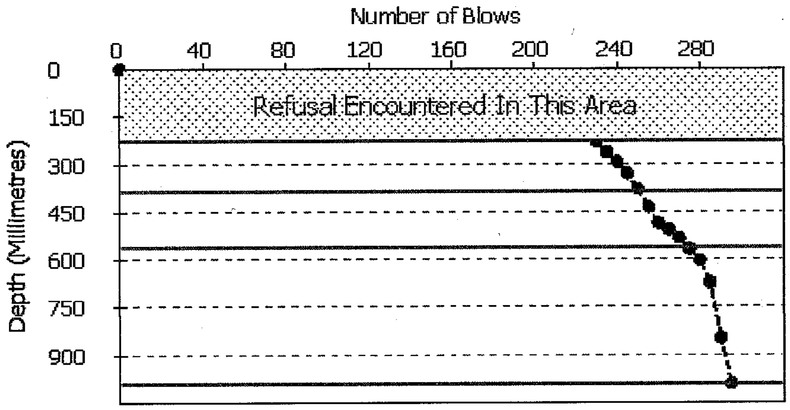
SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	C1	Cemented crushed stone or gravel	UCS**** : 6,0 to 12,0 MPa at 100 % Mod. AASHTO; Specification at least G2 before treatment; Dense - graded ; Maximum aggregate 37,5 mm
	C2	Cemented crushed stone or gravel	UCS : 3,5 to 6,0 MPa at 100 % Mod. AASHTO; Minimum ITS ***** = 400 kPa at 95 - 97 % Mod. AASHTO compaction; Specification at least G2 or G4 before treatment; Dense - graded; Max. aggregate 37,5 mm; Max. fines loss = 5 %******
	C3	Cemented natural gravel	UCS : 1,5 to 3,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 250 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 20 %
	C4	Cemented natural gravel	UCS : 0,75 to 1,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 200 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 30 %
	BEM	Bitumen emulsion Modified gravel	Residual bitumen: 0,6 - 1,5 % (SABITA, manual 14, 1993); Minimum CBR = 45 and Minimum UCS = 500 kPa @ 95 % Mod. AASHTO. Compaction: 100 - 102 % Mod. AASHTO
	BES	Bitumen emulsion Stabilized gravel	Residual bitumen 1,5 - 5,0 % (SABITA, manual 14, 1993); Minimum ITS***** = 100 kPa; Minimum resilient modulus 1000 kPa. Compaction: 100 - 102% Mod. AASHTO
	BC1	Hot - mix asphalt	LAMBS; Max. size 53 mm (SABITA, manual 13, 1993)
	BC2	Hot - mix asphalt	Continuously graded; Max. size 37,5 mm
	BC3	Hot - mix asphalt	Continuously graded; Max. size 26,5 mm
	BS	Hot - mix asphalt	Semi - gap graded; Max. size 37,5 mm
	AG	Asphalt surfacing	Gap graded (TRH 8, 1987)
	AC	Asphalt surfacing	Continuously graded (TRH 8, 1987)
	AS	Asphalt surfacing	Semi - gap graded (TRH 8, 1987)
	AO	Asphalt surfacing	Open graded (TRH 8, 1987)
	AP	Asphalt surfacing	Porous (Drainage) asphalt (SABITA, manual 17, 1994)
	S1	Surface treatment	Single seal (TRH 3, 1996)
	S2	Surface treatment	Multiple seal (TRH 3, 1996)
	S3	Sand seal	See TRH 3, 1996
	S4	Cape seal	See TRH 3, 1996
	S5	Slurry	Fine grading
	S6	Slurry	Medium grading
	S7	Slurry	Coarse grading
	S8	Surface renewal	Rejuvenator
	S9	Surface renewal	Diluted emulsion
	WM1	Waterbound macadam	Max. size 75 mm; Max.PI of fines = 6; 88 - 90 % apparent relative density
	WM2	Waterbound macadam	Max. size 75 mm; Max.PI of fines = 6; 86 - 88 % apparent relative density
	PM	Penetration macadam	Coarse stone + keystone + bitumen
	DR	Dumprock	Upgraded waste rock, maximum size 2/3 layer thickness

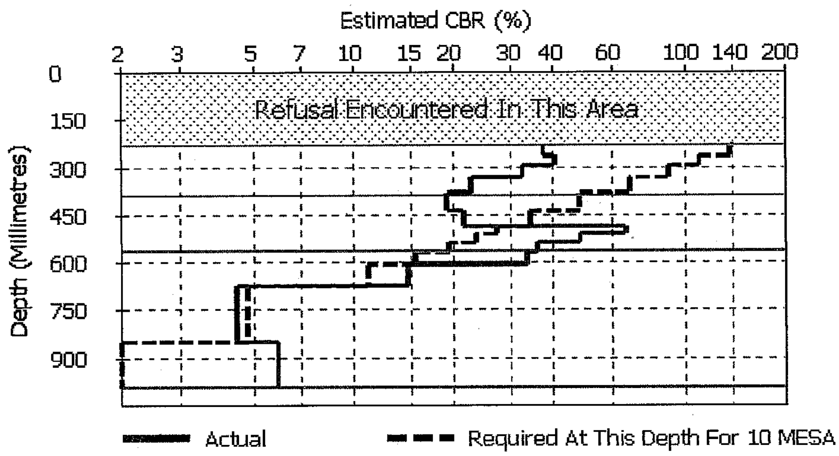
**** UCS: Unconfined Compressive Strength (TMH 1, 1979, Method A14)

***** ITS: Indirect Tensile Strength (SABITA Manual 14, 1993)

APPENDIX B

Dynamic Cone Penetrometer data





Layer Property Summary

Thickness (Millimetres)	Avg. Penetration Rate (Millimetres/Blow)	Estimated CBR (%)	Estimated Stiffness (MPa)
230	N/A	N/A	N/A
160	7.5	32	57.6- (131) - 300
175	7.75	30	55.6- (127) - 289
430	18.2	10	22.4- (51.1) - 117

Total Penetration Summary

Estimated Pavement Capacity	General Notes
26.3 MESA if Dry 12.3 MESA if at Optimum Moisture 5.7 MESA if Wet 2.7 MESA if Saturated	Blows to penetrate 800 mm = 289 Penetration Rate to CBR conversion is based on the relationship published by Kleyn (60 Deg. Cone)

TP1 (Km 1.0 Eastbound)

TP1 East (Note: For refusal area, 1mm/blow was assumed)

A-1

Rubicon Toolbox: DCP Analysis / Ver: 1.7 / (Licenced)

APPENDIX C

The use of FWD deflections in the rehabilitation design of flexible pavements Paper by Dr G T Rhode from Research Report 93/296, Department of Transport.

SECTION 8

THE USE OF FWD DEFLECTIONS IN THE REHABILITATION DESIGN OF FLEXIBLE PAVEMENTS

Prepared by G T Rohde
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8.1 INTRODUCTION

During the last few decades non-destructive deflection testing has become an integral part of the structural evaluation of pavements. From the many static, vibratory, and impulse devices the Falling Weight Deflectometer (FWD) has evolved as one of the favourite and most suitable devices for pavement evaluation (Lytton et al, 1987) [8.1]. The test method is rapid, relatively cheap, and the test equipment has been designed to subject a pavement to an impulse similar to that applied by a moving wheel load (Hoffmann and Thompson, 1981) [8.2]. This section of the rehabilitation course deals with the use of the FWD in rehabilitation investigations.

The state of the art in pavement evaluation is not yet at a stage where a single method of analysis can be used with full confidence. Ideally a pavement analyst should use and rely on a number of evaluation techniques in a multi-criterion approach to determine a pavement's overall structural and functional condition, possible problems and to select optimum maintenance and rehabilitation strategies (SARB, 1992) [8.3]. The role of deflection testing in a rehabilitation analysis is described in this section.

During the initial phases of a rehabilitation investigation deflection testing can be used effectively to select uniform sections and to strategically place test pits and hence laboratory tests. The interpretation and analysis of deflection data, like the deflection measuring devices, has gone through continuous improvements during the last two decades. Most of the available interpretation and analysis techniques fall into two categories; ie deflection basin parameters and the backcalculation of layer moduli.

The deflection basin parameters are used directly to evaluate a pavement's structural integrity (SARB, 1992) [8.3]. The parameters are derived either from the magnitude of the measured deflections, or the shape of the deflection basin (See Section 7). These parameters are empirically related to structural capacity or remaining life, and provides a direct measure of structural capacity. The backcalculation of layer moduli is founded on mechanistic principles. It allows for the evaluation of individual layers and provides information to identify the causes of distress in a pavement system especially on a project level investigation. Because it deals with fundamental material properties this method can be used to estimate future performance. The influence of the environment and the effect of changing wheel loads can also be investigated via the mechanistic method (also see Section 10). The deflection analysis procedures available for use in Southern Africa will be described and illustrated through worked examples. Finally this section is concluded by addressing some key issues in the use of deflection and its interpretation in pavement evaluation.

8.2 THE ROLE OF FWD TESTING IN REHABILITATION DESIGN

Rehabilitation design and pavement analysis have not yet reached a stage where a single method of analysis can be used with confidence. A holistic approach whereby analysts employ various test methods and analysis procedures are typically followed, ie the so-called multi-criterion approach. The results of various methods are compared to obtain an overall picture of a pavement's condition, expected life, and upgrading options.

A rehabilitation investigation normally progresses through a number of stages: First a condition assessment, then a structural capacity analysis, and finally a rehabilitation design and an economic analysis. As graphically illustrated in Figure 8.1 a cost-effective evaluation procedure should be followed. On the initial visit to a road, problem areas, distress types and the general composition of the pavement structure should be recorded. This should be followed by a detailed visual survey. Based on the initial overview, the available construction records, and visually identified uniform sections, the testing frequency of non-destructive deflection testing should be determined. The deflection results and observed areas of distress should then be used to determine the position and frequency of further, more expensive and labour-intensive in situ laboratory testing.

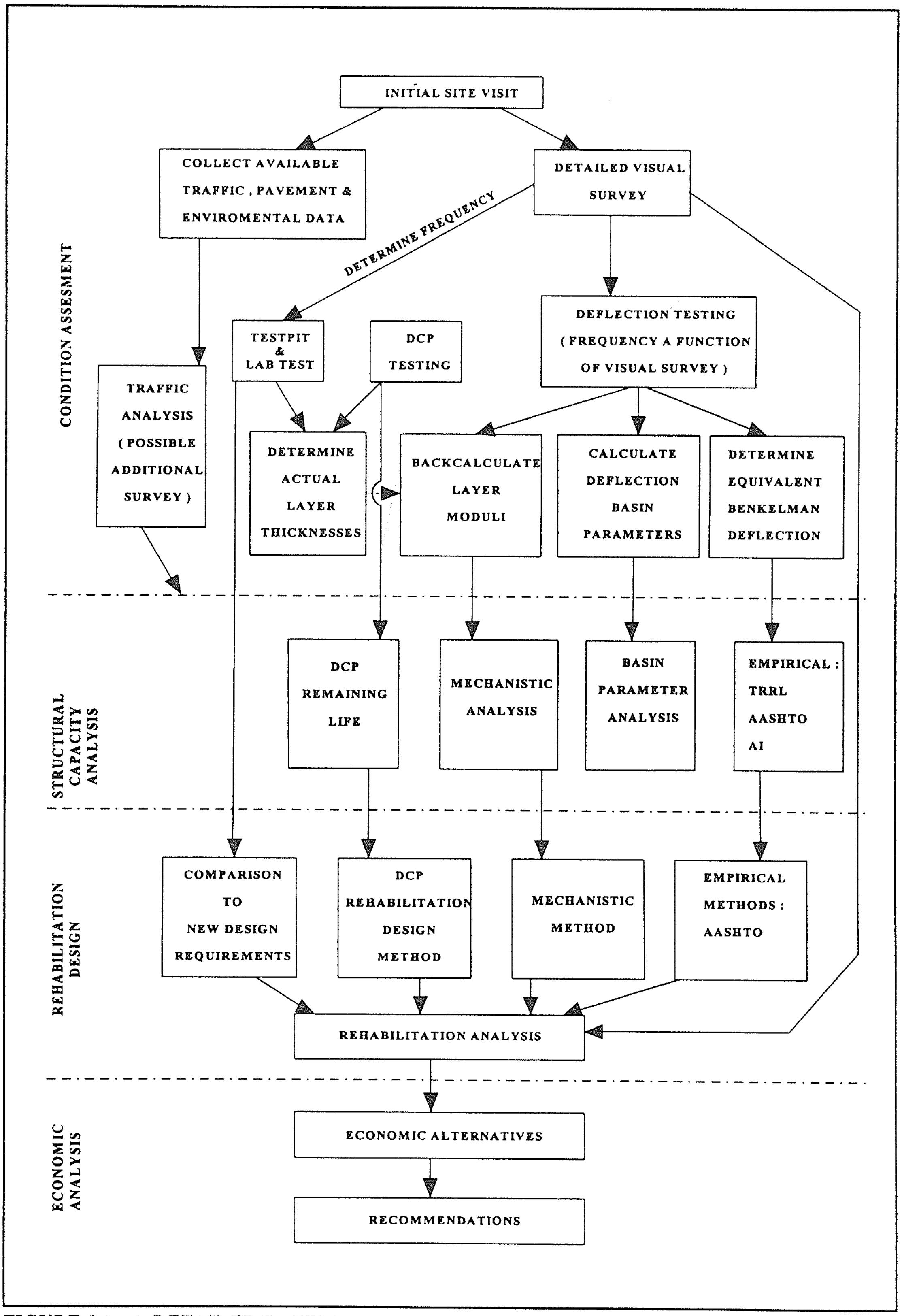


FIGURE 8.1 : A DETAILED PAVEMENT EVALUATION AND REHABILITATION DESIGN

According to SARB, 1992 [8.3], each method of assessment provides the designer with a different set of information. Therefore, the most complete assessment of any pavement can be made when considering the results of all available test methods. This approach is called the multi-criterion approach.

The three most commonly used non-destructive pavement assessment methods are: visual surveys, FWD measurements and DCP measurements. These could be augmented by test pitting and laboratory testing. Table 8.1 indicates the type of information that can be gained from each of these test methods.

TABLE 8.1 : INFORMATION OF PAVEMENT CONDITION PROVIDED BY DIFFERENT TEST METHODS (SARB 1992) [8.3]

INFORMATION PROVIDED	TEST METHOD		
	VISUAL SURVEYS	FWD	DCP
Surface Condition/Capacity	√√	√	X
Structural Capacity based on;			
1. Shear Strength	√	√	√√
2. Layer Stiffness	X	√√	√
3. Visible Distress	√√	X	X
Presence of Weak Layers	√	√	√√
Pavement Strength Balance	X	√	√√

√√ = Good Indication
√ = Some Information
X = No Information

From Table 8.1 it is clear that none of the three test methods alone provide all the information needed to carry out a detailed rehabilitation design. However, when the results of all three methods are considered together, not only is a more complete picture of the pavement condition or structural state provided, but the results of individual tests can also be verified to some extent. Furthermore, when two test results are in clear contradiction with one another, this may indicate the need for a more detailed analysis (such as test pits and materials testing), which would often show the presence of other factors which would most often influence the overall rehabilitation design.

- The factors that affect pavement performance are numerous and cannot be quantified or detected by any one test method.

- The results of some tests can be used to verify or dispute the results of other tests.

- Simultaneously considering the results of visual surveys, FWD and DCP measurements should provide a much more complete picture of the pavement condition, as each test method provides specific information on the pavement condition.

The structural capacity analysis and rehabilitation design should therefore be based on various analysis techniques. Within the assumptions and limitations of each method, the results should be compared to acquire effective rehabilitation alternatives. During this phase of the investigation deflection results can be analyzed in numerous ways to determine a pavement's structural capacity, determine problem layers and investigate strengthening options. An economic analysis should finally be conducted to determine whether a rehabilitation proposal is feasible and to select the most cost effective life-cycle rehabilitation strategy.

8.3 EMPIRICAL ANALYSIS TECHNIQUES UTILIZING FWD DATA

It is a well known phenomenon that the shape of a measured deflection basin is strongly influenced by the relative stiffness and properties of the pavement structure being tested, as was indicated in Section 7. Typically deflections measured at the outer sensors of a FWD are a function of the subgrade while the deflections recorded at sensors close to the load are influenced by the stiffness of the subgrade and the pavement structure above the subgrade. Through the years a number of deflection basin parameters have been defined to directly evaluate a pavement's structural integrity (Horak 1988). These parameters are derived either from the measured deflections, or the shape of the deflection basin. The most commonly used is the maximum deflection. Table 7.1 in Section 7 shows some typical deflection basin parameters used. In an empirical deflection analysis the deflection parameters measured on a pavement are evaluated relative to empirical standards or "tolerable" levels.

The measured basin parameters are not a generic property of a pavement system and at best, they can only be empirically related to pavement strength. As a result these relationships are only valid for the environment and type of pavement structures for which they were developed. Furthermore the deflection basin parameters are dependant on the type of measuring device. A relationship developed for a specific deflection device may not be applicable for use on deflection data obtained using a different device, because of the different referencing systems employed.

In the area of empirical deflection analysis two major applications for FWD collected deflections exist. The first involves relating the peak FWD deflection to an "equivalent" Benkelman Beam deflection and subsequently utilizing evaluation procedures traditionally developed for use with Benkelman Beam deflections. The second involves the use of several other basin parameters for which empirical standards have been defined for use in Southern Africa.

8.3.1 EQUIVALENT BENKELMAN BEAM DEFLECTION

Lacante (1992) [8.4] and De Beer (1992) [8.5] has investigated the relationship between the deflection basins measured under a Benkelman Beam type device and a those measured using a FWD. The results of this study found a linear relationship between the deflections measured with the two devices. The relationship summarized in Table 8.2, can be used effectively to determine the equivalent Benkelman Beam deflection from FWD test results. On large rehabilitation projects this relationship should be validated by conducting a few FWD and Benkelman Beam tests in parallel. For smaller projects the recommended ratios listed in Table 8.2 should be used.

The Benkelman Beam deflection is a parameter that can be used in several design procedures to determine the structural capacity of a tested structure. Of the procedures available those of the TRRL (Kennedy and Lister, 1978) [8.6] and the Asphalt Institute (Asphalt Institute, 1989) [8.7] are the most widely used. It should be kept in mind that the RSD is a modified Benkelman Beam and not exactly similar to the traditional Benkelman Beam (Coetzee and Bofinger 1988) [8.8].

TABLE 8.2 : CONVERSION FACTORS BETWEEN RSD MAXIMUM DEFLECTION AND FWD MAXIMUM DEFLECTION [8.4, 8.5]

TYPE OF BASE	RATIO (RANGE) : (δ (FWD)/δ (RSD))	RATIO (AVERAGE) (δ(FWD)/δ(RSD))
GRANULAR*	0,84 to 0,94	0,89
CEMENTED**	0,79 to 0,85	0,82
ASPHALT***	0,61 to 1,10	0,71 (@25°C)
Example : δ (FWD) = 307 μm; Ratio = 0,82 => δ(RSD) = (307/0.82) = 374,4 μm		

* Surface treatments (15 mm) up to 50 mm asphalt surfacing
** 50 mm to 60 mm asphalt surfacing
*** Temperature range : 20°C to 40°C; Asphalt thickness 150 mm to 200 mm

The Transport and Road Research Laboratory (TRRL) [8.9] has developed a structural capacity analysis technique to predict remaining pavement life from the maximum deflection measured under a Lacroix Deflectograph. To use this procedure with FWD deflections the maximum FWD deflection should first be translated to an equivalent beam deflection after which the beam deflection should be related to an equivalent deflectograph deflection. For this purpose the TRRL Report includes a correlation between Benkelman beam and deflectograph deflections. Some details of the procedure is documented in Section 3, Paragraph 3.2. It should be noted that this procedure is not appropriate to pavements with cement-treated materials.

The Asphalt Institute (AI) [8.7] also developed a deflection based analysis procedure where the remaining pavement life is a function of Benkelman Beam deflection and past traffic load. Details of this procedure is provided in Paragraph 3.3 of Section 3 and can be employed after the FWD deflection has been used to determine an equivalent beam deflection.

Recently the California Department of Transport (CALTRANS) published an overlay design procedure that can be used with the equivalent beam deflection (Forsyth et al, 1989) [8.10]. Figure 8.2 shows the relationship between the measured beam deflection, expected traffic load and the "tolerable deflection". If the measured deflection exceeds the tolerable level a procedure to reduce deflections through pavement strengthening is also available.

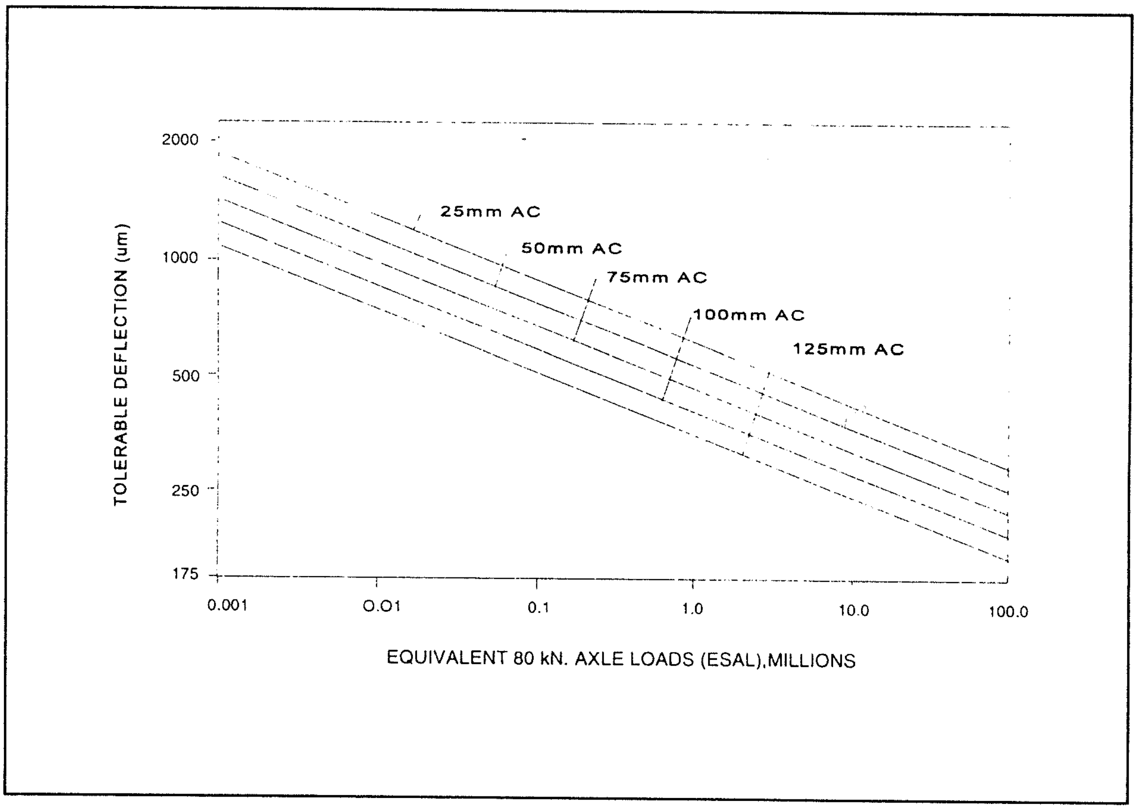


FIGURE 8.2 : THE CALIFORNIA DOT OVERLAY DESIGN PROCEDURE (Forsyth et al, 1989) [8.10]

8.3.2 DEFLECTION BASIN PARAMETERS

In Southern Africa a number of deflection basin parameters are used to evaluate measured FWD deflections. The parameters most commonly used are listed in Table 8.2. The parameter Y_{MAX}, the peak deflection measured under the FWD is influenced by the stiffness of the pavement structure and the underlying subgrade. The magnitude of this parameter provides an indication of overall structural capacity. The magnitude of the Base Layer Index (BLI) is indicative of the stiffness of the surfacing, base and sometimes the subbase. Typically, the magnitude of BLI strongly correlates with the stiffness of the top 200 mm of the pavement structure. The Middle Layer Index (MLI) is typically influenced by the stiffness of material at a depth of between 200 and 400 mm, and normally constitutes the subbase and selected layers. The radius of curvature (RC) is strongly related to the expected horizontal strain in an asphalt surface layer and thus the expected life to cracking of this layer.

TABLE 8.3 : MOST COMMONLY USE FWD DEFLECTION BASIN PARAMETERS FOR USE IN SOUTHERN AFRICA (SARB, 1992) [8.3]	
BASIN PARAMETER	**FORMULA**
Y_{MAX} - Peak Deflection (μm)	D_0
BLI - Base Layer Index (μm)	$D_0 - D_{300}$
MLI - Middle Layer Index (μm)	$D_{300} - D_{600}$
RC - Radius of Curvature	$\dfrac{(D_0 - D_a)^2 + r^2}{2\,(D_0 - D_a)}$
D_x - Measured FWD Deflection at an Offset of x mm as caused by a 40kN Impulse Load	
D_a - FWD Deflection at plate edge	
r - Radius of FWD Loadplate (Typically 150 mm). Since the FWD does not measure a deflection at an offset of 150 mm this deflection should be interpolated (see Eq. 8.5).	

In a research study sponsored by the South African Roads Boards (SARB, 1992) [8.3] relationships between the measured parameters and remaining life were developed. The following general relationship was suggested:

$$(\log Y) = B_0 + B_1 (\log N) \qquad \textbf{(Eq. 8.1)}$$

where
- Y = Measured Deflection Parameter in μm
- B_0,B_1 = Regression Coefficients as listed in Table 8.4
- N = Structural Capacity in E80s

TABLE 8.4 : DEFLECTION BASIN CRITERIA (SARB, 1992) [8.3]			
BASIN PARAMETER	**B_0**		**B_1**
	50%	**90%***	
Y_{MAX}	4,089	4,018	-0,235
BLI	3,960	3,857	-0,263
MLI	3,736	3,613	-0,274
RC	0,223	0,385	+0,288

* The SARB research suggests the use of the 90th percentile relationship on pavements that has experienced high traffic levels.

To appreciate the power and limitations of this procedure, a short description of the research background is provided. Actual deflection parameters were measured on 27 pavement structures. For each pavement structure tested, a structural capacity was defined according to the Draft TRH4 (1996) [1.7] catalogue of designs. The assigned capacity was adapted to account for variations in subgrade condition, material properties and layer thicknesses. The adapted structural capacities were related to measured basin parameters to obtain the relationships suggested in Table 8.4. It should be noted that a fairly wide scatter in data (prior to manipulation was observed). The pavements tested included pavements of various ages and conditions with various levels of past traffic. The measured deflection parameters were related to remaining pavement life (as first constructed). Therefore the structural capacities derived from Equation 8.1 is indicative of pavement capacity at the time of construction, and should be adjusted for traffic loads that have already used the facility by the time of deflection testing. Alternatively, as suggested in SARB, (1992) [8.3] lower 90th percentile transfer functions to compensate for past traffic. Another shortcoming of the approach is the fact that the pavements were tested throughout the year. No adjustments for temperature or seasonal variations were made. Some recommendations based on observations made overseas are made in SARB, 1992 [8.3].

To utilise the method, the following procedure is recommended:

1. Normalise deflections to 40 kN load levels. (Normalization is described in Paragraph 8.6.1.)

2. Calculate basin parameters as defined in Table 8.2.

3. Adjust the parameters for temperature and seasonal changes. It is suggested that the deflection basin parameters at 25°C and at equilibrium moisture condition be used. (Adjustments are described in Paragraph 8.6.2 and in Appendix C).

4. Determine the structural capacity using Equation 8.1 for each basin parameter.

5. Determine the remaining pavement life. This should be the difference between the structural capacity estimated above (using the 50th percentile relationship) and the estimated past traffic (in E80s) since first construction, or ignore past traffic and use the lower 90th percentile transfer functions, as suggested in SARB (1992) [8.3].

8.3.3 RADIUS OF CURVATURE

In 1989 Jung [8.11] published a procedure to directly calculate the maximum curvature and strain in asphalt layers under FWD testing. He suggested the following relationship:

$$R = \frac{(Y_o - Y_a)^2 + a^2}{2\,(Y_o - Y_a)} \qquad \textbf{(Eq. 8.2)}$$

where

	R	=	Radius of Curvature (mm)
	Y_o	=	Maximum FWD Deflection
	Y_a	=	FWD Deflection at the plate edge (Typically a = 150 mm)[1]
	a	=	Plate radius (mm)

From the radius of curvature the tensile strain at the bottom of the asphalt layer can be directly calculated using the following relationship:

$$\epsilon = \frac{H_1}{2R} \qquad \textbf{(Eq. 8.3)}$$

[1] Interpolating between the measured FWD deflections is required to determine the surface deflection at r = 150 mm. It should be noted that the deflection curve close to the loadplate is curved and linear interpolation will lead to an underprediction in deflection (see Eq. 8.5).

where ϵ = ... Horizontal tensile strain in the asphalt

H_1 = Thickness of the asphalt layer (mm)

R = Radius of curvature (mm)

Since the FWD load closely simulates a moving wheel load, the magnitude of the tensile strain under the FWD load should strongly correlate with the remaining fatigue life of the asphalt surface under normal traffic. Currently, no transfer functions to relate the FWD induced horizontal tensile strain to number of equivalent vehicle loads, is available for use in Southern Africa.

8.3.4 WORKED EXAMPLE

To illustrate the use of the empirical analysis techniques, deflection data collected on MR159 between Somerset West and Kuilsriver in the Cape Province was analyzed. The pavement structure of this road consists of

50 mm	Continuously Graded Asphalt
180 mm	Crushed Rock Base Course
250 mm	Crushed Rock Subbase
300 mm	Selected Material

Nondestructive pavement deflections were collected in 1992 and a pavement surface temperature of 26°C was measured during testing. The measured FWDs are listed in Table 8.5.

TABLE 8.5 : DEFLECTIONS AS MEASURED ON MR159									
POSITION (km)	LOAD		MEASURED DEFLECTIONS (µm)						
	kPa	kN	D_0	D_{200}	D_{300}	D_{600}	D_{900}	D_{1200}	D_{1500}
4.0	594	42	261	179	131	66	49	41	36
4.5	580	41	342	237	175	89	57	45	29
5.0	580	41	404	253	173	81	57	49	38
5.5	608	43	184	122	84	37	24	18	12
6.0	580	41	617	408	264	76	49	40	32
6.5	580	41	681	444	285	79	41	33	21
7.0	580	41	639	393	260	85	48	37	28
7.5	580	41	555	356	238	93	57	44	28
8.0	594	42	545	383	276	132	86	62	35

The first step in the analysis is to normalize all deflections to the standard 40 kN deflections (see paragraph 8.6.1). The results is shown in Table 8.6.

TABLE 8.6 : NORMALIZED DEFLECTIONS - MR159									
POSITION (km)	LOAD		MEASURED DEFLECTIONS (µm)						
	kPa	kN	D_0	D_{200}	D_{300}	D_{600}	D_{900}	D_{1200}	D_{1500}
4.0	566	40	249	171	125	63	47	39	34
4.5	566	40	334	231	171	87	56	44	28
5.0	566	40	395	247	169	79	56	48	37
5.5	566	40	173	115	79	35	23	17	11
6.0	566	40	600	397	257	74	48	39	31
6.5	566	40	668	435	279	77	40	32	21
7.0	566	40	620	381	252	82	47	36	27
7.5	566	40	546	350	234	92	56	43	28
8.0	566	40	522	367	265	127	82	59	34

The third step in the analysis process is to calculate the basin parameters as defined in Table 8.2. The results are shown in Table 8.7. Since the deflections were collected at a pavement temperature of 26°C no temperature corrections are necessary. (If required Table 8.16 would have been appropriate.) The next step in the parameter analysis is the calculation of remaining life. Since MR159 has experienced significant traffic loads the 90th percentile relationship suggested in Table 8.4 was used. The results are also listed in Table 8.7.

TABLE 8.7 : CALCULATED BASIN PARAMETERS AND STRUCTURAL CAPACITY FOR MR159

POSITION (km)	BASIN PARAMETERS (μm)				REMAINING LIFE IN E80s			
	Y_{MAX}	BLI	MLI	RC	Y_{MAX}	BLI	MLI	RC
4.0	249	124	62	238	8E06	5E06	4E06	8E06
4.5	334	163	84	182	2E06	2E06	1E06	3E06
5.0	395	226	90	126	1E06	5E05	1E06	9E05
5.5	173	94	44	320	4E07	1E07	2E07	2E07
6.0	600	343	183	92	2E05	1E05	9E04	3E05
6.5	668	388	202	80	1E05	7E04	6E04	2E05
7.0	620	368	170	78	2E05	8E04	1E05	2E05
7.5	546	312	143	95	3E05	2E05	2E05	3E05
8.0	522	258	138	120	3E05	3E05	2E05	8E05

Finally, the remaining life can also be calculated in terms of years to failure. The final results are shown in Table 8.8.

TABLE 8.8 : REMAINING PAVEMENT LIFE FOR MR159 BASED ON AN EMPIRICAL ANALYSIS				
POSITION (km)	REMAINING PAVEMENT LIFE (YEARS)			
	Y_{MAX}	BLI	MLI	RC
4.0	15+	15+	15+	15+
4.5	15+	15+	15+	15+
5.0	15+	10-15	15+	15+
5.5	15+	15+	15+	15+
6.0	3-5	0-3	0-3	5-10
6.5	0-3	0-3	0-3	3-5
7.0	3-5	0-3	0-3	3-5
7.5	3-5	3-5	3-5	5-10
8.0	3-5	5-10	3-5	15+

The table indicated possible failure between km 6 and 7.

8.4 MECHANISTIC ANALYSIS TECHNIQUES UTILIZING FWD DATA

Mechanistic analysis techniques refer to the use of analytical models to relate fundamental material properties and load characteristics on a pavement to its responses such as stress, strain and deflection. The use of the mechanistic models in utilizing measured deflections in a rehabilitation design are threefold. Firstly the models are used to analyze the measured pavement deflections. The effective modulus of elasticity of each pavement layer is determined from the measured deflections. This process is generally referred to as "backcalculation of layer moduli" (Lytton 1989) [8.12]. Secondly, the backcalculated layer moduli are used to mechanistically determine remaining pavement life (See Section 7). Analytical models are used to calculate stress, strain or deflections in a multi-layered system. These structural responses are then related to expected performance through the use of transfer functions. Thirdly, the same models can be used to investigate strengthening options (rehabilitation) by predicting pavement performance after adding or reworking pavement layers. Each of these procedures is described in some detail.

8.4.1 BACKCALCULATION OF LAYER MODULI

In the mechanistic analysis of pavement deflections the measured deflections and load characteristics are used to determine a theoretical model in terms of material properties and layer thicknesses that can explain the measured deflection basin. The mechanistic model selected can range from fairly simple to extremely sophisticated. Simple models such as Boussinesq's one layer or Burmister's two layer model are popular. The majority of "back-calculation" programs currently available use multi-layered elastic theory to derive a model to explain the measured deflections. Examples of these programs are MODULUS, ELMOD and BOWLER (See Section 7 and Appendix D). Currently, efforts at research level are concentrating on the use of finite element and dynamic analysis models to better utilize FWD data (Magnuson and Lytton, 1993) [8.13].

The mathematical models used in the layered elastic programs can determine pavement responses under surface loads with known pavement properties. However in the analysis of deflection data, the load and pavement responses (ie deflections) are measured (known) and the pavement properties are unknown. The backcalculation of layer moduli thus uses the layered elastic programs in a reverse (backward) fashion. This process has been described in detail in Section 7. A few key issues are repeated here:

1. Backcalculation requires expertise and engineering judgement. Pavements being tested are considerably more complicated than the relatively simple multi-layered elastic model being used to explain the measured deflections. It cannot be expected that the multi-layer elastic model will be able to explain/model all tested pavement sections.

2. The backcalculated layer moduli should be viewed as "model properties" and not "material properties". They are model dependant and do not necessary correlate well with laboratory moduli which were determined under different stress, moisture and load conditions.

3. The backcalculated moduli should be realistic. This is essential because more than one combination of layer moduli can lead to the same deflection basin (ie there is not a unique solution).

4. The moduli of layers with thicknesses less than 70 mm cannot be determined effectively (FWD plate radius = 150 mm).

5. The number of layers (n) in the model should be less than the number of deflections (n_d) being matched in the deflection analysis ($n_d = n+1$).

6. The moduli determined through backcalculation are only applicable to the moisture, temperature and stress condition at the time of testing. To determine the moduli at reference conditions the moduli should be adjusted (see Paragraph 8.6.2 and 8.6.3 and Appendix C).

7. Do not average two or more deflections prior to backcalculation. Backcalculate first and then determine statistical parameters of the backcalculated moduli.

8. Deflection analysis procedures are very sensitive to layer thicknesses. Determine the actual layer thicknesses as reliably as possible. For this purpose DCP tests are very useful.

9. On sections with shallow bedrock, or sandy subgrades, a model with an infinitely thick subgrade will lead to totally unrealistic moduli (Rohde, Scullion and Smith 1992) [8.34].

10. Moisture saturated layers in the undrained condition also lead to unrealistic high moduli derived from impulse load testing (See Figure 8.3).

11. Moduli of pavement layer material (unbound) may also be load frequency dependent (see Figure 8.3). More research however is needed to quantify this aspect better.

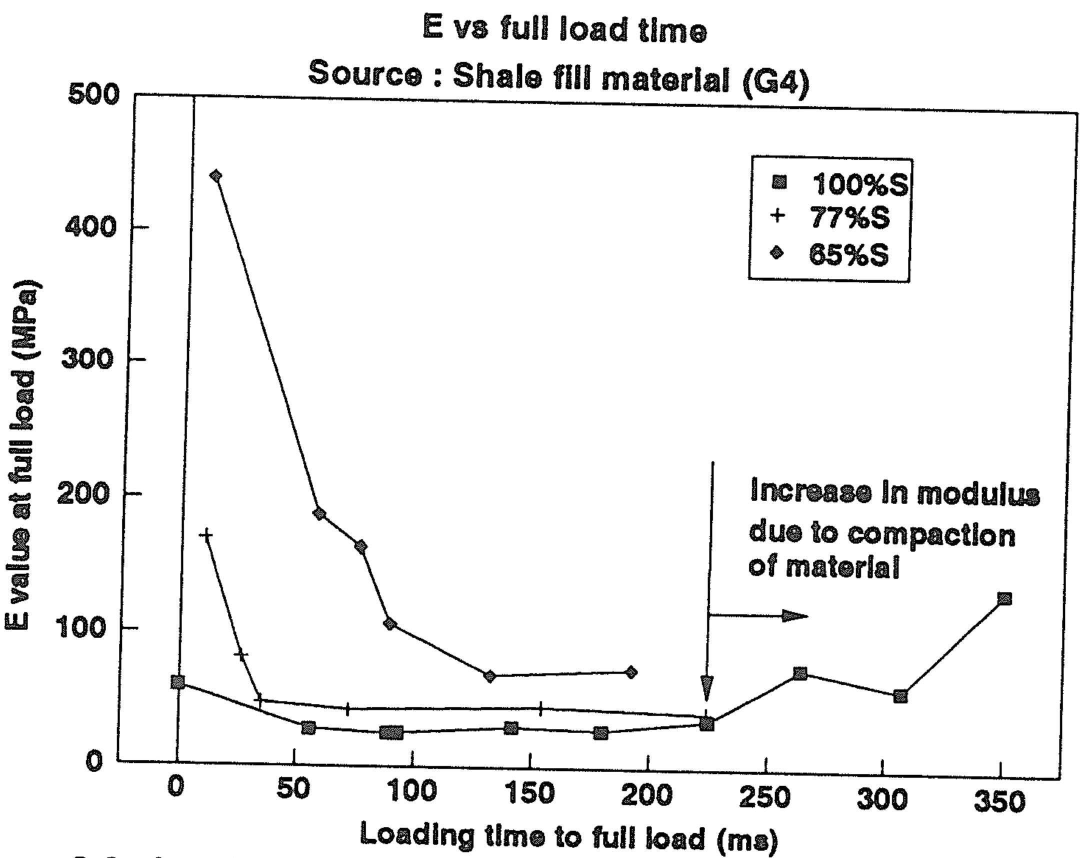


Figure 8.3 : Load frequency and moisture dependency of the elastic moduli of pavement layers found during K-mould testing

8.4.2 MECHANISTIC STRUCTURAL CAPACITY ANALYSIS

In a mechanistic analysis as described by Thompson (1989), stresses, strains and deflections induced by a wheel load are theoretically calculated in a multi-layered pavement system. These pavement responses are transformed into pavement performance parameters such as cracking or rutting, using performance models and empirical relationships, also referred to as transfer functions (See Section 6).

In using mechanistic design procedures for pavement design, it is important to utilize a close-loop approach (Thompson, 1989) [8.14]. Materials testing and evaluation concepts, structural modelling, climatic models etc. used in the design and development of the transfer functions should be similar to those used in the deflection analysis. This important principle is substantiated by Lytton (1989) [8.12]:

> *"The most common property found by NDT is the elastic stiffness of each layer. The method chosen (elastic modulus or the properties of the nonlinear stress-strain curve) should be compatible with the method that is used to make design calculations (multilayered or finite element methods). For consistency, the same method should be used to predict remaining life, to monitor the change of layer properties with time, and for use in specification testing."*

In terms of the existing deflection analysis techniques, this has several implications. If a layered elastic program is to be used in analysing pavements during design, then a layered elastic or equivalent technique should be used to analyze the deflection data. All assumptions made during backcalculation should be consistent with the pavement model used in rehabilitation design. This includes assumed layer and subgrade thicknesses, and material behaviour assumption (ie, linear elastic or stress sensitive) and type of deflection device to be used.

Compatibility between the transfer functions and the analysis model used is essential. Prior to the use of a transfer function it is therefore essential to ensure that assumptions and models used when devising the transfer function are compatible with the model used to analyze the deflections and calculate critical responses.

Locally and internationally, numerous transfer functions have been developed. Unlike the empirical type of relationships described in Paragraph 8.3 these functions are more transferable between environmental regions. Table 8.10 lists the critical responses that can be determined to predict performance or remaining life (See also Table 6.14 in Section 6). More detail of the functions are provided in Section 6 (South African Models) and in Appendix 8A of this section (International Models). It should be noted that the South African transfer functions for the performance of granular and stabilized materials were developed under a slow moving wheel load (creep speed) and are therefore not readily usable in a FWD analysis. Research to develop FWD compatible transfer functions for granular and stabilized material is being planned. Most international transfer models for subgrade stress and asphalt fatigue are compatible with the linear elastic analysis of FWD data.

To determine the remaining life of a pavement structure utilizing mechanistic principles the following procedure is recommended:

1. Normalize FWD deflections to a 40kN load level (See Paragraph 8.6.1). This procedure is automatically done in IDMP (see Appendix E).
2. Backcalculate the pavement layer moduli (See Section 7).
3. Adjust moduli for seasonal and temperature variations (See Paragraph 8.6.2 and 8.6.3 and Appendix C).

TABLE 8.10 : CRITICAL RESPONSES AND TRANSFER FUNCTIONS USED IN THE STRUCTURAL CAPACITY ANALYSIS

NO	MATERIAL TYPE	DISTRESS	REFERENCES	CRITICAL PARAMETER	FUNCTION
1	Asphalt	Fatigue Cracking	Asphalt Institute (1989) [8.28] Verstraeten et al (1982) [8.15] Thompson and Cation (1986) [8.16] Powell et al (1984) [8.17] Ullitdz (1977) [8.18] Fin et al (1986) [8.19] Shell (1992) [8.31] Potter and Donald (1985) [8.20] Freeme (1983) [8.21]	Horizontal Tensile Strain (Bottom of Layer)	$\epsilon_t = aN^h$
2	Granular	Shear Failure	Maree (1978) [8.22] Wolff (1992) [8.23]	Stress State (Middle of Layer)	$N = f(\sigma_1, \sigma_3, c, \phi)$ $N = f(\theta, \text{material})$
3	Stabilized	Fatigue Cracking	De Beer (1992) [8.5]	Horizontal Tensile Strain (Bottom of Layer)	$N = F(\epsilon_s/\epsilon_b)$
4	Subgrade	Permanent Deformation	Asphalt Institute (1989) [8.28] Powell et al (1984) [8.17] Shook et al (1982) [8.24] Potter and Donald (1985) [8.20]	Vertical Compressive Strain (Top of Subgrade)	$\epsilon_v = aN^h$

σ_1, σ_3 - Principal Stresses
N - Remaining life in standard E80s
ϵ_s/ϵ_b - Strain Ratio
ϵ_v - Vertical Strain

θ - Bulk Stress
c - Cohesion
ϕ - Friction
a, b - Constants
ϵ_t - Horizontal Tensile Strain

analyzed pavement system and assigning the resilient modulus of a new asphalt or high quality granular material to the layer. If an existing layer is reworked, an improved modulus is assigned to the reworked material. Critical responses such as those listed in Table 8.10 are calculated for the improved pavement system. The critical responses are related to remaining pavement life, which in turn is compared to the expected traffic.

8.4.4 WORKED EXAMPLE

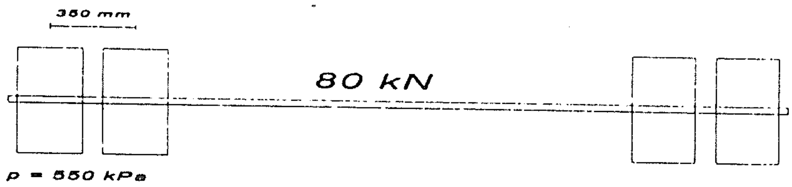
The FWD deflections reported in the worked example in paragraph 8.3.4 can also be analyzed mechanistically. Firstly all deflections should be normalized. This has been done in the previous example (See Table 8.6). The next step is the mechanistic analysis of measured deflection basins. The backcalculated moduli resulting from a deflection analysis are shown in Table 8.11.

TABLE 8.11 : RESULTS OF MECHANISTIC DEFLECTION ANALYSIS MR159

POSITION	BACKCALCULATED MODULI (MPa)				FIT (%) (ERROR/ SENSOR)
	SURFACE	BASE	SUBBASE	SUBGRADE	
4.0	3800	750	600	165	4.6
4.5	2900	600	300	150	4
5.0	2700	480	275	195	6.4
5.5	4500	950	790	200	6.7
6.0	4000	80	570	85	11
6.5	4500	100	195	104	6.8
7.0	4000	100	380	85	6.6
7.5	4500	150	250	138	4.6
8.0	2850	360	200	85	4.5

The backcalculated moduli were subsequently adjusted to equivalent moduli at 25°C. Although no adjustments in asphalt moduli are, strictly speaking, required between 25 and 26°C, Equation 8.14 has been applied in this example to the asphalt moduli in Table 8.11. The corrected moduli are shown in Table 8.12.

The backcalculated moduli were subsequently subjected to a mechanistic analysis to determine the critical stresses and strains under a standard 80 kN axle load. This standard design load is defined as:



In this analysis two critical pavement responses were investigated. The first is the tensile strain that develops at the bottom of the asphalt layer. (This response can be associated with fatigue cracking.) The second response is that of vertical stress on the subgrade (The response associated with permanent deformation). The critical responses and associated remaining pavement life is provided in Tables 8.12 and 8.13. The following two transfer functions were used in this analysis.

$$\textit{For asphalt fatigue}: N = 10^{\left(\frac{3.561 - \log \epsilon t}{0.189}\right)}$$

$$\textit{For subgrade } \Sigma v = 0{,}0029 * N^{0.10}$$

TABLE 8.12 : CRITICAL RESPONSES OF MR159						
POSITION	TEMPERATURE CORRECTED MODULI (MPa)				ϵt ($\mu\epsilon$)	ϵv ($\mu\epsilon$)
	SURFACE	BASE	SUBBASE	SUBGRADE		
4.0	3860	750	600	165	142	202
4.5	2950	600	300	150	177	260
5.0	2750	480	275	195	204	234
5.5	4580	950	790	200	115	162
6.0	4070	80	570	85	417	422
6.5	4580	100	195	104	384	473
7.0	4070	100	380	85	379	464
7.5	4580	150	250	140	298	357
8.0	2900	360	200	85	231	426

TABLE 8.13 : REMAINING PAVEMENT LIFE					
POSITION	CAPACITY (E80S)		PAST TRAFFIC	REMAINING LIFE	
	N_{ac}	N_{sg}	(E80s)	Surface	Subgrade
4.0	3E+07	4E+11	4.7E+05	2E+07	4.0E+11
4.5	9E+06	3E+10	4.7E+05	8E+06	3.3E+10
5.0	4E+06	9E+10	4.7E+05	4E+06	9.2E+10
5.5	9E+07	3E+12	4.7E+05	9E+07	3.3E+12
6.0	1E+05	2E+08	4.7E+05	0	2.6D+08
6.5	1E+05	7E+07	4.7E+05	0	8.3E+07
7.0	2E+05	9E+07	4.7E+05	0	9.9E+07
7.5	6E+05	1E+09	4.7E+05	9E+04	1.4E+09
8.0	2E+06	2E+08	4.7E+05	2E+06	2.3E+08

From the above calculation it is evident that MR159 has reached failure between km 6 and 7.5 while the first 4 kilometres have adequate structural capacity, which is similar to the findings based in surface deflection basin (Paragraph 8.3, Table 8.8).

8.5 DERIVING A PAVEMENT'S STRUCTURAL NUMBER FROM FWD TESTS

The Structural Number (SN) is used as an indicator of pavement strength in a number of pavement design and performance prediction models. The deflections measured using the FWD can be used effectively to determine a pavement's structural number and characterize the subgrade strength. Currently two techniques can be used effectively. The first uses the shape of the deflection basin to estimate the structural number and subgrade modulus. This procedure has been documented in detail by Rohde (1994). The method is summarized in the following steps:

1. Normalize measured FWD deflections to standard 40 kN load deflections.
2. Determine the deflection at an offset of 1.5 times the total pavement thickness. This will require interpolation between deflections measured at the fixed sensor positions. To interpolate on a curve fitted through 3 fixed positions the following relationship can easily be programmed:

$$D_X = \frac{(R_X - R_B)(R_X - R_C)}{(R_A - R_B)(R_A - R_C)} D_A + \frac{(R_X - R_A)(R_X - R_C)}{(R_B - R_A)(R_B - R_C)} D_B + \frac{(R_X - R_A)(R_X - R_B)}{(R_C - R_A)(R_C - R_B)} D_C$$

(Eq. 8.5)

where: D_x = Deflection at an offset of R_x

D_i = Deflection at sensor i

R_i = Offset of sensor i

i = A, B, C being the 3 closest sensors to point X

x = Point for which deflection is determined.

3. The effective structural number, at the temperature and moisture condition during testing can be determined using the following relationship:

$$\mathbf{SN_{eff} = k_1 (D_o - D_{1.5Hp})^{k2} Hp^{k3}} \qquad \textbf{(Eq. 8.6)}$$

where SN_{eff} = Structural Number (Effective)

D_i = Measured deflection in µm at an offset of i

Hp = Total pavement thickness in mm

k_1 = 0,1165 for seals ; 0,4728 for asphalt surfaced pavements

k_2 = -0,3248 for seals ; -0,4810 for asphalt surfaced pavements

k_3 = 0,8241 for seals ; 0,7581 for asphalt surfaced pavements

It is important to note that the calculated structural number is relevant for the prevailing temperature and moisture condition at the time of deflection testing. To determine the structural number at a standard temperature the peak deflection, D_o, should be corrected to an equivalent peak deflection at the reference temperature. For this purpose the procedure described in Section 8.6.2 should be used.

To obtain an estimate of the subgrade modulus the following steps should be used:

1. Determine the surface deflection at offsets of 1.5 x Hp and 1.5 x (Hp + 300). This should be interpolated using Equation 8.5.

2. Calculate the subgrade stiffness using the equation:

$$E_{sg} = 10^{k4} (D_{1.5Hp} - D_{1.5(Hp + 300)})^{k5} Hp^{k6} \qquad \text{(Eq. 8.7)}$$

where
E_{sg} = Subgrade Modulus in MPa
D_i = Measured FWD deflection at an offset of i
Hp = Total Pavement Thickness in mm
k_4,k_5,k_6 = A Coefficient as defined in Table 8.14

TABLE 8.14 : COEFFICIENTS FOR USE IN EQUATION 8.7			
TOTAL PAVEMENT THICKNESS (mm)	k4	k5	k6
< 380	9,138	-1,236	-1,903
380 - 525	8,756	-1,213	-1,780
> 525	10,655	-1,254	-2,453

3. For use in design procedures where the subgrade strength is defined in terms of in situ subgrade strength, the following relationship (Emery, 1985) [8.25] can be used as transformation:

$$E_{sg} = 41.19\ CBR_u^{0.385} \qquad \text{(Eq. 8.8)}$$

where
E_{sg} = Subgrade Modulus in MPa
CBR = California Bearing Ratio (Unsoaked)

Secondly a pavement's structural number can be derived from the backcalculated layer moduli. For this purpose the following equation, suggested by AASHTO (1986) [8.26] should be used:

$$SN = \frac{1}{25.4} \sum_{i=1}^{n} h_i \, a_g \left(\frac{E_i}{E_g} \right)^{1/3}$$ **(Eq. 8.9)**

where a_g = Layer coefficients of standard materials used in AASHO road test

E_g = Resilient modulus of standard materials in the AASHO road test

h_i = Layer thickness (mm)

SN = Structural Number

Although Equation 8.9 is more accurate than Equation 8.6 it requires exact knowledge of layer thicknesses, is time consuming, and relies heavily on backcalculation expertise. The AASHO Road Test consisted of the following three materials:

Asphalt Concrete : a = 0,44

Crushed Rock Base : a = 0,14

Granular Subbase : a = 0,11

Currently a number of design procedures rely on the structural number as strength parameter.

8.5.1 AASHTO ANALYSIS PROCEDURE

The 1986 AASHTO pavement design guide (AASHTO, 1986) [8.26] relates the thickness and stiffness of each pavement layer to design traffic in terms of equivalent 80 kN axle loads. The following relationship is used:

$$\log(E_{80}) = Z_R\, S_O + 9{,}36 \log(SN+1) - 0.2 + \frac{\log\left[\dfrac{\Delta\, PSI}{4{,}2-1{,}5}\right]}{0{,}4 + \dfrac{1094}{[SN+1]^{5{,}19}}} + 2{,}32 \log(M_R) - 8{,}07$$

.....(Eq 8.10)

where:

Z_R	=	Standard normal deviation
S_o	=	combined standard error (Default 0.35)
SN	=	$a_1D_1m_1 + a_2D_2m_2 + a_2D_3m_3$
a_i	=	i^{th} layer coefficient
d_i	=	i^{th} layer thickness
W_{18}	=	predicted number of E_{80} load repetitions
M_R	=	Resilient Modulus in psi
ΔPSI	=	Allowable reduction in serviceability from measured PSI
m_i	=	Moisture coefficient

In a rehabilitation investigation Equation 8.10 can be used to determine the structural number required to accommodate the expected traffic. The difference between this required structural number and that measured on the pavement through deflection testing can be used to decide on strengthening options. The following steps should be followed:

1. Measure deflections and determine SN_{eff} and E_{SG} using Equations 8.6 and 8.7 above.
2. Determine the SN required to accommodate the expected traffic using Equation 8.10.
3. Determine the strengthening required:

$$h_{st} = 25{,}4 \left[\frac{SN - SN_{eff}}{a_{st}} \right] \qquad \textbf{(Eq. 8.11)}$$

where	h_{st}	=	Thickness required of the selected material to increase SN_{eff} to SN
	SN	=	Structural Number required
	SN_{eff}	=	Effective Structural Number of pavement to be strengthened
	a_{st}	=	Structural coefficient of material to be used in strengthening. Typical coefficients are listed in Table 8.15.

TABLE 8.15 : TYPICAL LAYER COEFFICIENTS a_i		
LAYER	**MATERIAL**	**COEFFICIENT**
Surface	Asphalt Stability 4 Stability/Flow 2	0,44
Base	Bituminous Waterbound Macadam G1 G5	0,35 0,22 0,20 0,14
Subbase	G5 G7	0,12 0,11

8.5.2 TRRL CATALOGUE OF DESIGNS (ROAD NOTE 31 - 1993) [8.9]

The effective structural number, determined from surface deflections measured on a pavement, can also be evaluated against the TRRL's catalogue of designs. This catalogue, captured in Road Note 31, is based on TRRL experience in many countries of which several are located in Southern Africa. An extract of the catalogue is shown in Figure 8.3. The following procedure should be followed:

1. Determine from the catalogue the required pavement design for the relevant traffic and subgrade conditions. The in situ subgrade strength required to select a design, can be obtained from a deflections analysis.

2. Determine the structural number of the required pavement structure.

3. Use the measured deflection to determine SN_{eff}.

4. Use Equation 8.11 to determine the thickness of an overlay or strengthening layers.

8.5.3 CPA CATALOGUE [8.33]

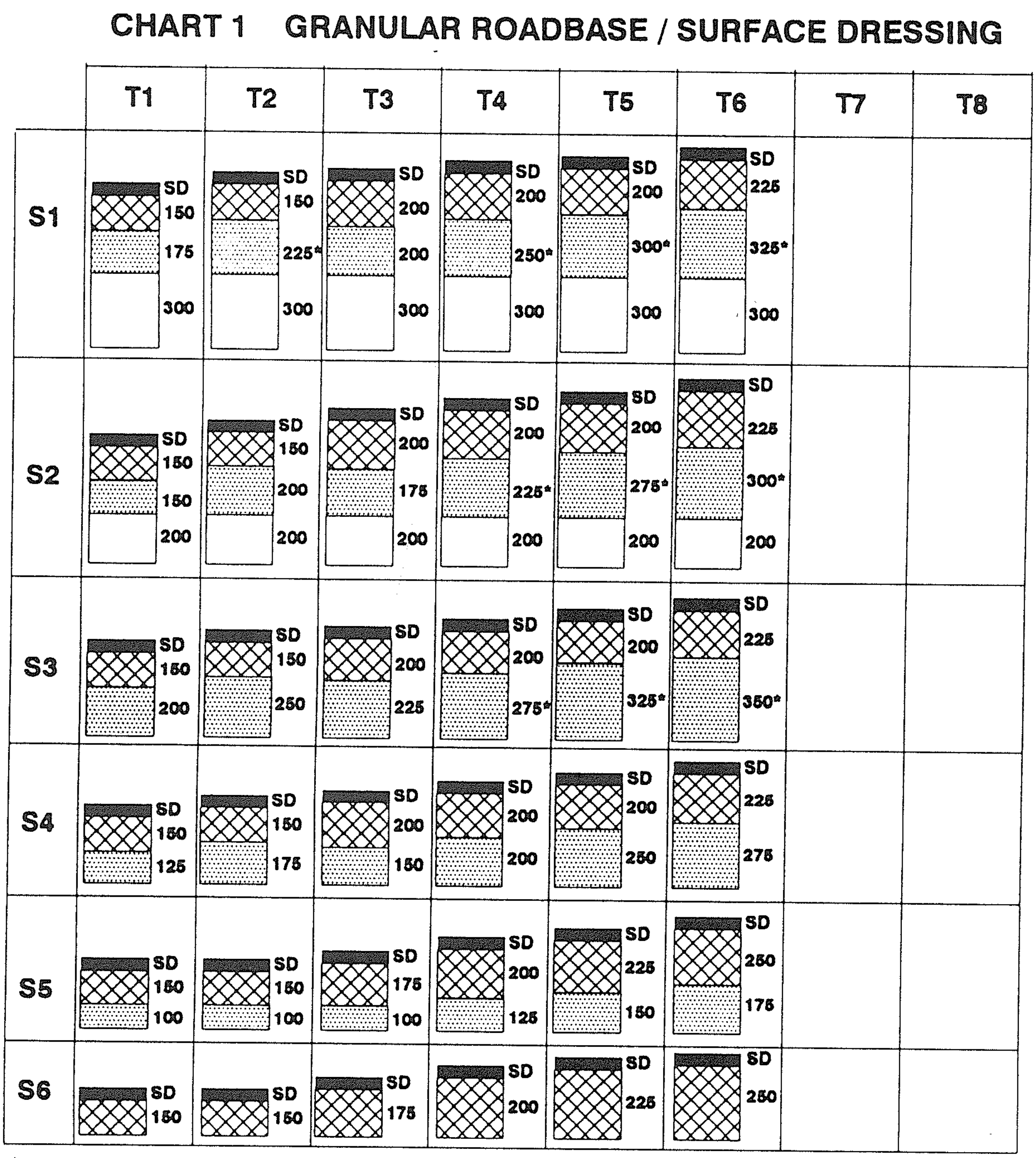
In the 1983 Materials Manual of the Cape Provincial Administration (CPA) [8.33] a catalogue of standard pavement designs has been published. These designs are based on local adaption of the original AASHTO Road Test Formula. An extract of the Catalogue is shown in Figure 8.4. Measured deflections and the catalogue can be used effectively in a rehabilitation investigation. The following procedure should be followed:

1. From the catalogue select a pavement design for the applicable region, traffic class and subgrade strength.

2. Calculate the structural number of the selected pavement design (SN required).

3. Use the measured deflection and Equation 8.6 to determine SN_{eff}.

4. Use Equation 8.11 to determine the thickness required to obtain a strengthened pavement equivalent to that required in the catalogue of designs.



Note : 1 * Up to 100mm of sub-base maybe substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm

2 A cement or lime-stabilised sub-base may also be used.

12310/aa.833

FIGURE 8.4 : TYPICAL DESIGN CATALOGUE AS EXTRACTED FROM ROAD NOTE 31 (1993) [8.9]

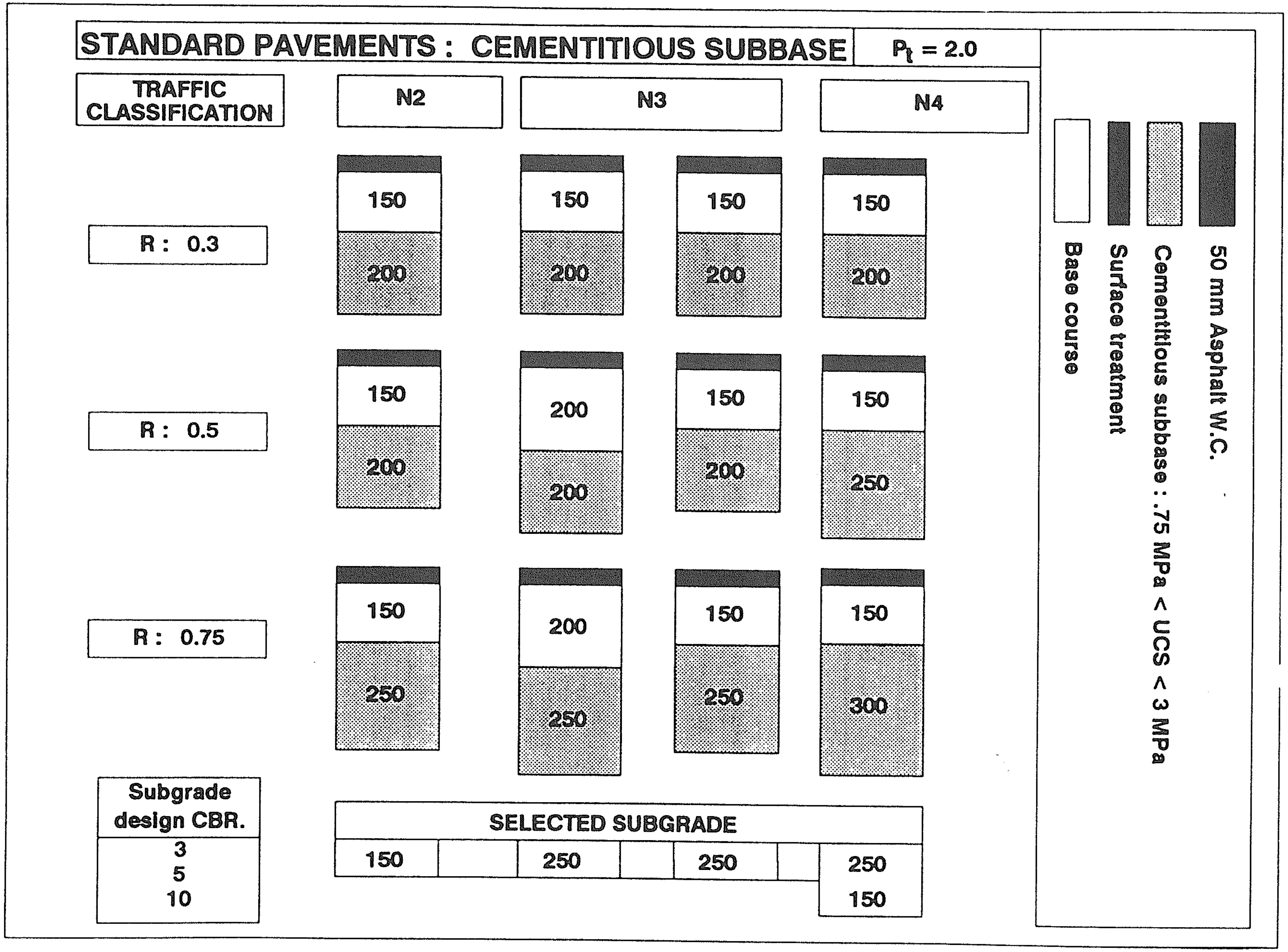


FIGURE 8.5 EXTRACT FROM CPA MATERIALS MANUAL [8.33]

8.6 OTHER DEFLECTION ISSUES

8.6.1 DEFLECTION NORMALIZATION

The magnitude of the FWD Impulse Load is a function of the drop height of the falling weights. Typically the instrument is set to discharge the weights at a height that will cause an impact of approximately 40 kN. Because this target load is never precisely met, the measured deflections should be "normalized" to exactly 40 kN. For this process it is assumed that the entire pavement system is linear elastic and all the deflections are factored using the following formula:

$$D_N = \frac{L_N}{L_M} \cdot D_M \qquad \textbf{(Eq. 8.12)}$$

where:

D_N = Normalized deflection (μm)
D_M = Measured deflection (μm)
L_N = Load to which normalization is done (Typically 40 kN)
L_M = Load during testing

Because pavement materials are not linear elastic it is important to conduct deflection testing as close to 40 kN load levels as possible.

8.6.2 TEMPERATURE CORRECTIONS

The stiffness of asphalt concrete is highly temperature dependant. This necessitates that deflections measured on asphalt pavements be corrected for temperature effects. A number of temperature correction procedures for backcalculated moduli are presently used. The first approximation is based on research conducted by the Asphalt Institute (1981, 1989) [8.7], [8.28] and subsequently recommended for correction of deflection results (Lytton et al, 1990) [8.27]:

$$LogE_{std} = logE_{field} + 0.028829P_{200}\left[\frac{1}{(f_o)^{\lambda}} - \frac{1}{(f)^{\lambda}}\right]$$

$$+ 0.000005\sqrt{P_{ac}}\ [(t_o)^{ro} - (t)^{r}]$$

$$- 0.00189\sqrt{P_{ac}}\left[\frac{(t_o)^{ro}}{(f_o)^{1.1}} - \frac{(t)^{r}}{(f)^{1.1}}\right]$$

$$+ 0.931757\left[\frac{1}{(f_o)^{n}} - \frac{1}{(f)^{n}}\right] \qquad \textbf{(Eq. 8.13)}$$

where:

Estd	=	AC concrete modulus at the standard temperature and frequency
E_{field}	=	AC concrete modulus at the prevailing field temperature and frequency
λ	=	0.17033
n	=	0.02774
t	=	test temperature (degrees Fahrenheit)
f	=	loading frequency (hertz)
t_o	=	standard temperature (77°F, 25°C)
f_o	=	standard frequency (5Hz)
P_{ac}	=	percent AC by weight of the mix
r_o	=	1.3 + 0.49825 log (f_o)
r	=	1.3 + 0.49825 log (f)
P_{200}	=	percent aggregate passing the No 200 sieve.

Detail of the above relationship is also given in Appendix C.

Johnson and Baus (1992) [8.29] simplified the above relationship by assuming on P_{ac} of 5,7% and the typical FWD load duration as approximately 30 to 40 ms:

$$\frac{E_{std}}{E_{field}} = 10^{-(0.0002175[(t_o)^{1.886} - (t)^{1.886}])} \qquad \textbf{(Eq. 8.14)}$$

Another correction, developed by Ullidtz (1987) [8.18], is based on backcalculation of moduli from AASHO Road Test deflections. This relationship for asphalt temperatures above 35°F is given by:

$$E(t) = 2.18 \ x \ 10^6 psi - 1.15 \ x \ 10^6 \ psi \ x \ \log \ (t°E)$$

(Eq. 8.15)

$$x \ 10^6 \ psi \ x \ \log \ t°C$$

where E(t) is the AC concrete modulus (pounds per square inch) at the test temperature (t) (degrees Celsius). After solving for E(25°C) = E_{std} and setting E(t) = E_{field}, Equation 8.15 may be rewritten as:

$$\frac{E_{std}}{E_{field}} = [3.319 - 1.752 \ \log \ t°C]^{-1}$$ **(Eq. 8.16)**

Equation 8.16 or 8.14 can be used to correct backcalculated moduli to the Standard design temperature.

A method to adjust calculated basin parameters for temperature variations has also been suggested by Jooste and Maree in SARB (1992) [8.3]. The correction factors suggested for deflections measured on granular base pavements with surfacings of 75 mm or less is shown in Table 8.16.

TABLE 8.16 : TEMPERATURE CORRECTION FACTORS FOR GRANULAR BASE PAVEMENTS WITH SURFACINGS OF 75 mm OR LESS* (SARB 1992)[8.3]		
TEMPERATURE	CORRECTION FACTOR	
	D_0	D_{200} AND D_{300}
> 35°C	0,9	0,95
15 - 35°C	1,0	1,0
10 - 15°C	1,1	1,05
< 10°C	1,2	1,1
* No factors are to be applied to Surface Treated Pavements		

8.6.3 SEASONAL DEFLECTION CORRECTIONS

Recent studies for the Department of Transport (Rohde, 1994) [8.30] clearly illustrates the seasonal variation in pavement deflections monitored on South African Roads. Although a procedure to adjust deflections for seasonal influences is still being developed some typical field results measured is shown in Figure 8.7.

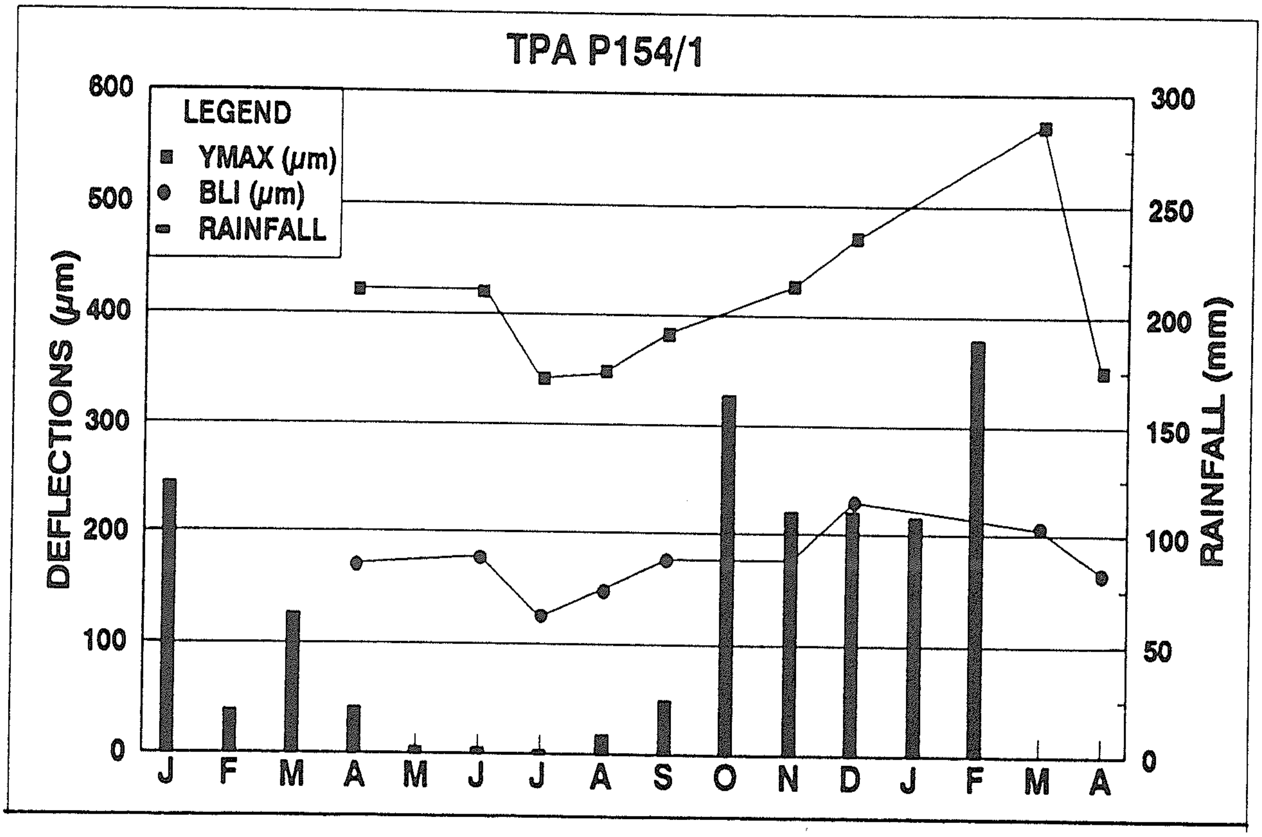


FIGURE 8.6 SEASONAL DEFLECTION VARIATION AS OBSERVED ON TPA ROAD 154/1 IN 1993/4

Pages 8-40 to 8-45 has been removed as irrelevant.

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APPENDIX 8A

Brief Description of a Number of Internationally used Mechanistic - Empirical Transfer Functions.

8A ASPHALT FATIGUE

Fatigue of asphalt concrete layers under repeated loading is an important factor in the design of flexible pavements. Internationally numerous relationships have been developed to relate the tensile strain in an asphalt layer under loads to the number of load repetitions to failure. A number of the available relationships are briefly described below. As emphasized by Gomez and Thompson (1984) [A.8.1] the relationships based on laboratory fatigue tests and field calibration are structural model dependant. It is strongly recommended that the background to the transfer functions be investigated prior to their use.

8.A.1. The Asphalt Institute

The fatigue equation utilized in the 1981 MS-1 thickness design manual (Asphalt Institute 1981) [A.8.2] is:

$$N = C * 18.4 \, (4.32 \times 10^{-3}) \, (1/\epsilon_t)^{3.29} \, (1/E)^{0.854} \qquad \textbf{(Eq. A1)}$$

where:

N = number of 18 kips-equivalent single axle loads for 20 per cent or greater fatigue cracking.

ϵ_t = maximum tensile strain in the asphalt layer (in/in)

E = asphalt mixture dynamic modulus (psi)

C = a correction factor equal to 10M

where:

$$M = 4.84 \left(\frac{V_b}{V_v + V_b} - 0.69\right) \qquad \textbf{(Eq. A2)}$$

in which:

V_b = volume of asphalt, percent

v_v = volume of air voids, percent

For a standard mix, the Asphalt Institute equation for 20% of area cracked is:

$$N_f = 0.0796\ \epsilon_t^{-3.291}\ E^{-0.854} \qquad \textbf{(Eq 8.A3)}$$

8.A.2. Belgium Road Research

The Belgium Road Research (Verstraeten et al, 1982) [A.8.3] has developed the following relationship:

$$N = (4{,}92 \times 10^{-14})\ \epsilon_t^{-4{,}76} \qquad \textbf{(Eq. A4)}$$

8.A.3. Illinois DOT

The Illinois DOT developed a strain-based fatigue algorithm for a dense-graded asphalt mixture (Thompson and Cation, 1986).

$$N = 5 \times 10^{-6}\ \epsilon_t^{-3.0} \qquad \textbf{(Eq. A5)}$$

This equation was established based on considerations of mixture composition factors, split strength characteristics, and field calibration studies.

8.A.4. TRRL

Powell et al, (1984) [A.8.4] developed AC fatigue cracking criteria based on an analysis of the field performance of several experimental flexible pavements. A multi-layer linear elastic analysis procedure was utilized to calculate dynamic strains. Fatigue relations used were of the form obtained by laboratory testing over a range of temperatures and levels of dynamic strain. Miner's hypothesis was utilized to accumulate fatigue damage. The curves derived in the laboratory required considerable adjustment to match observed road performances.

The fatigue algorithms with an 85 % reliability are:

$$N = (4.17 \times 10^{-10})\ \epsilon_t^{-4.16} \quad \text{(Dense graded Macadam)} \qquad \text{(Eq. A6)}$$
$$N = (1.66 \times 10^{-10})\ \epsilon_t^{-4.32} \quad \text{(Rolled Asphalt)} \qquad \text{(Eq. A7)}$$

8.A.5. Denmark

Ullitdz (1977) [A.8.5] presented the following fatigue relationship:

$$\mathbf{N = (3.4 \times 10^{-21})\,(V_b)^{5.62}\,(1/_t)^{5.62}} \qquad \textbf{(Eq. A8)}$$

V_b is percentage of bitumen by volume

According to Ullitdz the criterion was developed from laboratory fatigue tests, corrected for the influence of rest periods, and the allowable normal stress on the subgrade from an analysis of the WASHO Road Test, the AASHO Road Test and the CBR-curves. The values have been modified in accordance with experience gained during the last decade. The failure criteria underlying these allowable values are thus fatigue cracking of the asphalt layer and the minimum acceptable PSI-value."

8.A.6. NCHRP Project 1 - 10

Finn et al (1986) [A.8.6] suggested the following fatigue relationships:

For 10 % cracking:

$$\text{Log } N = 15.97 - 3.291 \text{ Log}\epsilon_t - 0.854 \log E \qquad \text{(Eq. A9)}$$

For 45 % cracking:

$$\mathbf{\text{Log } N = 16.086 - 3.291 \log \epsilon_t - 0.854 \text{ Log } E} \qquad \textbf{(Eq. A10)}$$

ϵ_t = AC tensile strain, microstrain

E = AC modulus, ksi

8.A.7. Shell

The Shell Pavement Design Manual recommends the following fatigue relationship:

$$N = 0.0685\, \epsilon_t^{-5.671}\, E^{-2.363} \qquad \text{(Eq. A11)}$$

8.A.8. Australian Road Research

In the 1985 Interim pavement design guide published by the Australian Road Research (Potter and Donald, 1985) [A.8.7] the following transfer function is documented:

$$\epsilon_t = 225 \left(\frac{N}{10^6} \right)^{-.5} \qquad \textbf{(Eq. A12)}$$

where: ϵ_t = Critical tensile strain at the bottom of the asphalt layer
N = Number of E_{80} repetitions to failure

8.B PERMANENT DEFORMATION

A number of relationships have been developed that relate vertical subgrade strain to the development of permanent deformation (rutting).

The relationships all have the following general form:

$$\epsilon_v = k_1 \; x \; N^{k_2} \qquad \textbf{(Eq. A13)}$$

where:

ϵ_v = Vertical Elastic strain on the subgrade in $\mu\epsilon$.
N = Number of Repetitions
$k_1 k_{,2}$ = Coefficients from Table 8B

TABLE 8B : COEFFICIENTS FOR EQUATION A13				
ORGANIZATIO N	FAILURE CRITERIA	K1 * 10^6	K2	REFERENC E
TRRL	10 mm Rut Depth	0,0145	-0,25	Powell et al (1984)
Nottingham	Excessive Deformation	0,0216	-0,28	Brown et al (1982)
Asphalt Institute	13 mm Rutting	0,0102	-0,22	Shook et al (1982)
NAASRA	20 - 30 mm Rutting	0,0085	-0,14	Potter and Donald (1985)
South Africa (see Eq. 6.29, 6.30 and 6.31)	8 mm 12 mm 18 mm	0,001279 0,002483 0,004375	- 0,0178 - 0,0872 - 0,1081	RP/2/81 RP/2/81 RP/2/81

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APPENDIX D

ASSESSING MATERIAL PROPERTIES FOR PAVEMENT REHABILITATION DESIGN

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Abstract

The use of Mechanistic-Empirical design methods is gaining popularity internationally with the development of the new US Mechanistic Empirical Pavement Design Guide (MEPDG), the new Caltrans Mechanistic-Empirical procedure (CalME) and the improved South African Mechanistic Design Method (SAMDM). These design methods incorporate ongoing improvements to pavement models and development and refinement of material test methods. These developments, combined with expanding pavement database information systems and entrenched, perhaps less sophisticated, but proven, design methods, leave the present day pavement design engineer with a dilemma as to how to assess material properties for rehabilitation design.

To diagnose pavement distress, characterise in-situ layer stiffness and calibrate failure models, designers are using a number of methods to assess the condition of pavement and individual layers that include: visual assessments, deflection measurements, DCP tests, material sampling and laboratory testing, as well as newer sophisticated measurements (ground penetrating radar, seismic measurements, etc).

This paper provides some insights into the specific properties that need to be measured, indications of typical test frequencies and best practices on interpretation of results and data analysis. The paper highlights the complexity of the determination of material properties, as well as the importance of adopting a holistic approach and understanding the principles of each test and material property.

1 INTRODUCTION

During the evaluation and design of a pavement, the pavement designer is confronted with a large number of possible tests and information that could be used. Available information ranges from riding quality and skid resistance measurements on a network level, to test pit excavations and laboratory test results from sampled materials at the project level, all of these often at different frequencies. Added to this, a variety of pavement evaluation and design methods are available, each developed with, and calibrated for a specific set of properties collected from specific test methods.

Although a number of methods are used to design pavements, the Mechanistic-Empirical (ME) methods are arguably the most accurate, if applied correctly. The worldwide trend is towards the increased use of these methods, e.g. the new US Mechanistic-Empirical Pavement Design Guide (MEPDG) and the South African Mechanistic Design Method (SAMDM). Most other methods, e.g. the Dynamic Cone Penetrometer (DCP) and

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deflection based overlay design methods, require limited input (only DCP tests or maximum deflections). On the other hand, the ME methods are data intensive and the material properties to be used are often difficult to obtain and not well understood. Different tests to obtain the same property required for ME design will produce different results, and the designer must understand how to interpret the results. This is complicated by the increased cost of testing and the consequential tradeoffs between data collection intensity and the risk of missing important information.

The aim of this paper is to review procedures and approaches used in obtaining material properties for use in pavement rehabilitation design and to provide information on best practices.

2 CURRENT PAVEMENT REHABILITATION DESIGN METHODS

Pavement rehabilitation design is a highly complex and integrated procedure, combining different types of information. Table 1 gives an indication of the types of information relevant to pavement (rehabilitation) design. This paper addresses only the last type, material properties, but designers MUST take cognisance of the other factors.

Table1: Relevant information in pavement rehabilitation design

Type of information	Comment
Traffic	Heavy vehicle axle loading in particular
Original pavement design and construction details	Important in understanding defects, deciding on rehabilitation options and uniform sections
Maintenance and rehabilitation records	Important in understanding defects, deciding on rehabilitation options and uniform sections, and identifying materials in place
Cross-section	Sections built as part of road-widening efforts have properties different from the original roadway
Climatic conditions	Important to understand past behaviour and to predict future performance
Geology and topography	Geology can have a large influence on pavement performance
Site constraints	Underground services, bridge clearances, etc. may dictate the rehabilitation option considered
Drainage	Presence of permeable shoulders, high water tables, etc. may warrant special measures to be taken and often explain the existing condition
Availability of materials	The availability or non-availability of certain materials should direct the rehabilitation options
Analysis and design procedure or method used	Methods depend on the level of sophistication required, skills available, and type of rehabilitation. These indicate the material properties to be obtained
Pavement material properties	To be discussed in detail in this paper

As mentioned earlier, one very important aspect in the selection of the material properties is the identification of the pavement analysis and design method to be used.

The design method can range from the selection of an appropriate pavement composition from a catalogue, to obtaining an overlay thickness from a deflection–overlay thickness relationship, to the determination of pavement layers providing a required structural number, to the use of a mechanistic-empirical process to determine the required pavement composition. The material property input required ranges from one value, e.g. CBR, DCP or deflection, to a multiple of properties, e.g. resilient moduli, Poisson's ratio and failure properties for each layer in different environmental situations.

The selection should be based on the procedures available, the traffic levels, and the type of pavement or rehabilitation envisaged. As an example, if materials will be used for the rehabilitated pavement that were not considered in the original development of the empirical design method, then a higher level of laboratory material testing and a more flexible (e.g. M-E) design method are likely to be required. On the other hand, if only an asphalt overlay is envisaged obtaining asphalt thicknesses from an empirical determined maximum deflection–thickness relationship may suffice. Similarly, roads with high traffic volumes warrant more extensive testing and analysis.

2.1 Southern African design methods

The methods used in South Africa for rehabilitation design are described in detail in TRH12 (TRH12, 1997) and include:

- DCP method (Jordaan, 1989)
- Deflection bowl parameter method (SARB, 1992)
- Maximum surface deflection (Jordaan, 1994b; Lester, 1972; AI, 1989)
- AASHTO method (AASHTO, 1993)
- SAMDM (Theyse et al, 1996).

Two levels of analysis are recommended in TRH12 (TRH12, 1997). The first is for a preliminary design where nothing more than the type, or types, of appropriate rehabilitation per uniform section is determined. The second entails a detailed analysis to determine the exact rehabilitation required..

The **DCP method** compares the penetration rate, DCP number (DN) from the existing pavement, with that of the required pavement to establish additional layer works needed. In some cases DN values of new layers are supplemented with grading and plasticity index (PI) requirements.

The **deflection bowl parameter method** does not consider any material properties of the layer and is solely based on measured surface deflections. The properties of the required overlay are not defined in detail and only the type of material (e.g. asphalt, crushed stone) and thickness are specified in the **maximum deflection method.**

The **ME design approach** is based on the assumption that the structural response in the pavement, i.e. stresses, strains and deflections induced by the loads, can be used to predict pavement distresses, i.e. primarily rutting and cracking. The elastic moduli of the pavement layer are the most important input in converting the induced loads to stresses, strains and deflections. Multi-layer elastic theory or finite element methods are used to determine the pavement response. Established transfer functions then relate the pavement response to the pavement distress. The ME design method is discussed in more detail in the next section.

a. International design methods

Internationally, various methods of asphalt pavement rehabilitation design have been introduced over the past 30 years (Monismith and Brown (1999)). All of these methods rely on back-calculated stiffness of the existing pavement materials. The back-calculation programs include BISAR, EVERCALC, EFROMD2 (from CIRCLY) and PADAL for the Shell, WSDOT, Austroads and Nottingham procedures respectively. Only the Nottingham method uses laboratory testing to help with characterisation of existing layer stiffness.

The Mechanistic Empirical Pavement Design Guide (MEPDG) (NCHRP, 2004; Thompson, 2006) is in the process of evaluation and potential implementation in the United States. The MEPDG is the result of a project of the National Cooperative Highway Research Program (NCHRP 1-37A), and is currently being considered by AASHTO states for implementation. The MEPDG considers a national spectrum of distresses in North America, including thermal cracking, fatigue cracking, unbound layers rutting, top-down longitudinal cracking, and asphalt mix rutting.

A set of mechanistic-empirical models called CalME is being developed in California that uses incremental-recursive analysis in which the properties of the layers are updated, based on damage calculations throughout the simulation (Ullidtz, 2006). CalME is being developed by the University of California Pavement Research Center (UCPRC) and includes improved models for distresses particularly relevant to California (reflection cracking, asphalt rutting, unbound layers rutting). Caltrans and several other states will be evaluating these models for implementation over the next year. New pavement design is considered in both the MEPDG and CalMe.

The MEPGD describes three levels of user input sophistication, ranging from Level 3 default values, to Level 1, where expensive detailed laboratory testing (primarily tri-axial testing) is called for. In practical terms, the UCPRC has found that the user will likely use a mix of Level 1, 2 and 3 inputs depending on the resources available (laboratory equipment, trained staff) and funding and schedule considerations. It would generally be very expensive to do a complete Level 2 or Level 1 design, and most designers will likely use at least some Level 3 inputs. The Level 2 inputs primarily consist of regression functions that use simpler, less expensive tests to estimate stiffness, and transfer functions needed for the calculations. Some of these are very useful, while for others the required "simple" inputs are not well aligned with standard testing practices, which make it difficult to provide the needed information for Level 2 input calculations.

CalME primarily relies on back-calculated materials properties (stiffness, transfer function coefficients) and operates on a database of past values that a user can retrieve if a similar material is to be used in the design. This is particularly useful for expensive asphalt tests, where a similar aggregate, binder and mixed design have been used previously and the previous laboratory testing results are applicable.

Consideration of the effects of the pavement structure on subgrade and granular base stiffness differs in the MEPDG and CalME. The MEPDG considers non-linearity of the subgrade by use of finite element analysis and laboratory tri-axial testing. The MEPDG calculates a simple subgrade stress sensitivity function as part of the back-calculation process (using CalBack) and uses that parameter in the recursive design calculations.

The MEPDG considers non-linearity of the base stiffness either by calculation of confining stresses using layered elastic theory and using an iterative process, combined with laboratory tri-axial test data, or by use of finite element analysis with tri-axial results.

Another widely used rehabilitation design method is the Austroads design method (Austroads, 2004a; Austroads, 2024b), which is also based on surface deflections and ME principles.

b. Overview of methods

2.1.1 Empirical design methods

Empirical design methods are developed by statistically correlating variables such as materials properties and types, climate and traffic with observed pavement performance. Pavement performance is typically defined as the number of standard axle load repetitions applied before "failure" occurs, with failure typically not distinguished between different distress mechanisms (i.e. cracking or rutting measured at the surface). Examples of such methods are design catalogues, DCP and AASHTO.

Reliability, meaning the probability that the pavement will not reach failure prior to the design axle load repetitions, can be determined if sufficient observations are available in the data set used to develop the method. Reliability is calculated from the variance of the observed performance around the performance equation.

Materials tests for these methods must be those used to develop the test method. Substitute tests can be used if sufficiently strong correlations are found between the original test and the substitute test. The range of that correlation (which materials, which conditions, etc) must be considered, as most statistical correlations in pavement engineering are highly non-linear.

2.1.2 The ME design method

The first step in ME pavement design is characterising materials, traffic and environment conditions for a mechanistic model. The complexity of information needed to define the pavement structure, particularly under different loads, temperature conditions and moisture states, has increased as knowledge of the material's response and the ability of computers to handle calculations have increased. The primary input required to characterise most materials for ME design is "stiffness".

The most common constitutive relation assumed for the ME pavement model is isotropic, linear elasticity for all materials; therefore, the primary materials characterisation information required for the pavement structural model is the "equivalent" linear elastic modulus. The equivalent linear elastic modulus is defined here as the elastic stiffness measured under a given state of stress/strain, temperature, load time duration, moisture condition (suction and water content), aging/curing and compaction. Differences in these conditions occur in the field depending on the age of the pavement, location of the testing (in the wheel path or out of the wheel path), season, recent rainfall, time of day, and testing equipment used. Differences in these conditions occur in the laboratory depending on the sampling and handling, conditioning of the material, and the test being used.

The critical stress, strain or deformation is correlated with observed performance in the field. For ME design each distress is analysed individually. A different critical response is correlated with performance for each distress mechanism. Materials characterisation for the performance model (also known as the transfer function) is determined by the design method developers, and not left to the designer. However, for some new materials, or where existing transfer functions have not done a good job of predicting performance, laboratory characterisation and reconciliation with existing transfer functions may be required.

Transfer functions can be developed from direct correlation of the critical stress, strain or deformation with observed field or accelerated pavement test performance; or they can be developed from laboratory tests, and then adjusted based on observed field or Accelerated Pavement Testing (APT) performance. Development of transfer functions from laboratory tests reduces the number of field sections or APT tests required to perform the calibration, and can be performed under a much wider range of conditions.

Transfer functions are subject to a number of conditions and it should be expected that the transfer function may not accurately predict field performance if the transfer function was calibrated for a different set of conditions than are expected on a project.

The form of a transfer function, the variables included in it, and the values for the coefficients, are based on:

- The materials characterisation during its calibration.
- The numerical engine used to calculate the critical responses.
- The performance criteria used for its calibration.

If the transfer function is used with a materials characterisation that does not produce similar stiffness measurements, different assumptions in the calculation engine (such as bonding), or was developed using a performance data set from conditions that are considerably different than those in which it is being applied, then it can lead to unreasonable estimates of pavement performance.

3 OTHER IMPORTANT CONSIDERATIONS

A number of other factors also influence the selection of the type of test, frequency, time of testing, and material property to be used in the pavement rehabilitation analysis and design.

3.1 Reliability and variability

Reliability in ME design methods can be handled in a number of different ways, including:

- **Incorporated into the design method transfer functions** by the method developers by considering the variance between predicted and observed performance in the calibration data set. The designer then selects the transfer equation with the desired reliability, but must understand that the variance in construction quality, natural materials, climate, design traffic prediction, etc are all assumed to be the same as in the calibration data set.
- **Calculated by the designer using Monte Carlo simulation**, with the designer providing or using default probability distributions for a few key variables, such as materials stiffness, rainfall and temperature regimes, etc. This approach requires more knowledge on the part of the designer, and to be done correctly requires measurement or estimation of the variability of the key parameters for the specific project.

One limitation of Monte Carlo simulation is that it usually assumes that each variable is independent, when in fact they usually are not. A conceptual example is shown in Figure 1. In this example, the asphalt compaction is often correlated with the asphalt thickness, with thicker asphalt layers producing better compaction, over a range of 25 to 35 mm. This is commonly observed in practice, particularly for large maximum aggregate sizes. Granular base compaction is inversely correlated with its thickness, with the thicker layers not receiving sufficient compaction because the compaction equipment isn't heavy enough to produce high shear stresses at the bottom of the layer. In this example, sub-grade and base compaction is correlated locally with a drainage problem which results in a wet subgrade, and therefore poor support, to the base during compaction. Cut and fill thicknesses will also influence the pavement performance due to compaction in fill areas, and interception of water table and different soil types in cut areas. This figure does not even consider lateral and vertical variability, because layered elastic theory cannot consider these.

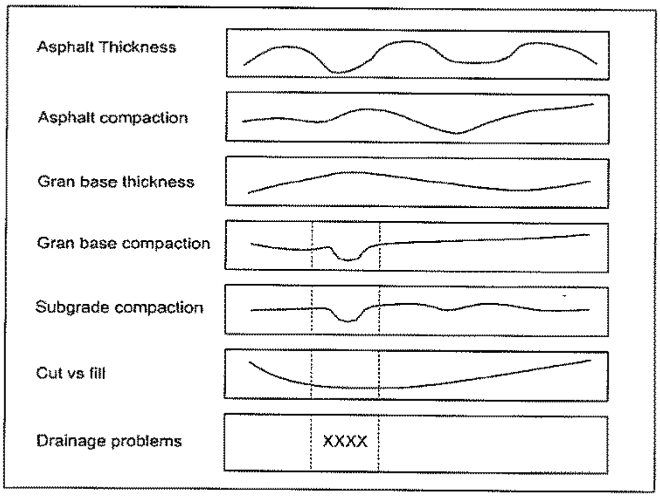

Figure 1: Conceptual presentation of variability along a project

These interactions provide an argument for using materials and thickness characterisation methods that provide complete coverage of the project length, and a density of measurements that can identify localised areas with distinctly different properties (such as the drainage problem), and interaction of the different variability patterns. For example, a designer should consider whether two or three CBR tests or tri-axial stiffness tests are adequate to characterise the project. Without knowledge of the variability, how would the sampling site be selected? What is the representative compaction to be used for the tri-axial specimen?

Characterisation methods that provide greater density of measurements, such as DCP, FWD for structural capacity and GPR for thickness, will provide an indication of these different variability patterns and their interactions. Observation of surface distresses will help identify where "perfect storms" of the variability occur, because these are locations where the pavement will begin to fail first.

Current research regarding reliability in ME design is investigating the interactions of the variability of different variables controlling pavement performance, and methods of using project characterisation data to develop designs with appropriate reliability.

3.2 Homogeneous sections

The selection of homogeneous sections is important, but not covered in detail in this paper. A homogeneous section can be defined as a section of road where specific properties are identical (e.g. traffic, age, surface type, original pavement design) or uniform (e.g. roughness, rutting deflections, visual condition). Three issues are relevant:

- Which property or parameter or combination is used to select uniform sections?
- Which method is used to define the sections i.e.
  - Constant length. This could be linked to practical rehabilitation or construction lengths.
  - Change point. (Kennedy, 2001).
  - Smallest common denominator (Dynamic segmentation method).
  - Peak standard deviation segmentation.
  - Cumulative sum method.
    - One of the last three is normally recommended. (Austroads, 2005a).
- Variability within each section. A coefficient of variation of less than 25% for the CBR of the subgrade in new pavement designs has been proposed.

3.3 Environmental effects

The influence of the environment on pavement performance and material properties is significant (Brown, 2004). The effect of the environment should be taken into account in the analysis and design procedure. A number of relationships and techniques have been developed to quantify the effect of moisture, and temperature in particular, on deflections and material properties (Hartman, A.M. (1996), Pufahl, D.E. et al (1990), Asphalt Institute (1989), Fwa, T.F. (2006), AI, 1986; Lytton, 1990; Henneman, (2007)). The effect of the environment can either be reflected in the adjustment of tests (typically deflection measurements) or material properties determined from tests.

3.4 Drainage

Condition survey and deflection of DCP information should identify localised areas with drainage problems.

The cost-effectiveness of drainage solutions for entire projects with extensive drainage deficiencies should be compared with the cost of pavement designs that assume materials properties with poor drainage. In some cases it may be more cost-effective to design for poor drainage. Great caution should be used when including drainage features, and designing with materials properties that assume that the drainage solutions that will be installed will be 100 percent effective. Maintenance of drainage features is a particular problem for some projects, and the cost of drainage maintenance, and the effects on materials properties if drainage is not maintained (for even one rainy season) should be considered in the risk analysis for different alternative designs.

4 MATERIALS PROPERTIES FOR DESIGN

4.1 Sources of information

4.1.1 Visual assessments

Visual assessments form the initial primary source of information in the decision to rehabilitate a section of road or not, and to distinguish between uniform sections. During detailed assessment the visual condition assessment data will again be used to verify rehabilitation design calculations, i.e. whether the location of a weak layer according to mechanistic analysis corresponds with the typical defects that would be observed on the road surface when that layer is no longer contributing to the structural integrity of the pavement system.

A number of procedures can be used to define and classify defects, i.e. HDM4 (Odoki, 1997) TRH12 (TRH12, 1997), TMH9 (TMH9, 1992) and Austroads (Austroads, 2005b).These documents also contain very useful discussions on the causes of the visual distresses. Table 2 summarises typical visual defects and their causes.

Table 2: Typical visual defects and their causes

Defect		Description
Cracking	Crocodile	• If traffic associated then deflections will be high, start inside wheel paths as longitudinal or map cracks, associated with rutting. • Dry brittle surface, i.e. just a function of the asphalt/seal. No rutting will be visible and cracks will usually be between the wheel paths.
	Block	Usually due to shrinkage of treated layers. The spacing is dependent on natural material and type or amount of treatment.
	Longitudinal	• If traffic associated then inside wheel path, next stage will be crocodile cracks. • If construction related then it will not be wheel path related but on joints or where segregation occurred. • If due to swelling of subgrade or embankment settlement then cracks will not be limited to wheel path but appear also on shoulders.
	Transverse	Usually temperature related, due to shrinkage of aged asphalt. Transverse cracks due to shrinkage of unbound materials very rare in Southern Africa.
Deformation	Corrugation	Surface distress related to asphalt mix properties or poor construction (paver or roller problems).
	Rutting	Confined to wheel path, but with different shapes.
	Undulation	Active subgrade or collapse settlement – environment issues, rutting not located in wheel paths only.

4.1.2 Riding quality and rutting

Different devices are used to measure road roughness of which profilometers provide absolute measurements. Measurements vary considerably depending on the wheel path travelled by the measuring wheel and averaged values (over a certain length) of the outer wheel path are usually reported.

Roughness can be expressed in a number of units (e.g. international roughness index, Present Serviceability Index (PSI), Quarter car index). Further issues of importance are: where the measurements should be taken (normally in both wheel tracks), over what section length the information should be summarised (normally 100 m lengths) and whether one or both wheel track measurements should be used

Rutting (surface deformation) can be measured manually or with electronic equipment as the deviation under a 1.2 m or 2 m straight edge. Continuous measurements are normally summarised, as an average, over 100 m lengths.

The riding quality and rutting properties are important in understanding the cause and mechanism of distress, and thus in the selection of appropriate tests, but cannot be used to directly determine material properties.

4.1.3 Skid resistance

Measurements can be obtained from the SCRIM, Griptester, sand patch, pendulum, mobile texture meter (MTM) or high speed lasers, and produce either micro or macro texture properties. Continuous measurements are normally summarised, as an average, over 100 m lengths. These properties do not have any relevance in the determining of material properties for rehabilitation design.

4.1.4 Dynamic cone Penetrometer (DCP)

The Dynamic Cone Penetrometer (DCP) measures the rate of penetration of a cone tipped steel rod through a pavement layer. The DCP test is non-destructive and relatively inexpensive and gives a measure of the relative shear strength of all layers to a depth of 800 mm. Consequently, DCP surveys are usually done as secondary tests during the detailed assessment phase, when measurements can be limited to a number of tests within each uniform pavement section. Due to logistical problems, it is often not possible to commission deflection testing in remote parts of Africa and in these cases the easily transportable DCP can provide structural data at an acceptable frequency relatively quickly as part of the initial assessment phase of the investigation.

DCP tests are not used for assessment of heavy cemented layers, asphaltic layers or crushed stone layers (max aggregate size > 37.5 mm) (Harvey et al, 2006). DCP data can be analysed to determine layer thicknesses (although difficulties in distinguishing between layer interfaces do exist), shear strength, equivalent unconfined compressive strengths and effective elastic modulus of pavement layers. (De Beer, 1989; Gabr, 2000); (Jordaan, 1989a). The analysis of granular layers requires some moisture adjustment, and moisture sampling during field testing is desirable. As an example, material with a high clay content can indicate false high strengths under dry conditions.

Testing is typically spaced at around five tests per km, staggered on the inner, outer and between wheel tracks, with a test point added to significantly defective areas (at least 8 DCP tests per uniform section). Usually a DCP test would be performed at every Test pit location before excavation of the Test pit to calibrate the measured penetration rates with moisture and laboratory CBR data. Whenever possible, DCP measurements should also be taken at the same locations where deflections are measured.

Devices that operate on the same principles as the DCP include the Rapid Compaction Control device (RCCD) and the Pocket Penetrometer.

4.1.5 Deflections

The main measures of pavement surface deflection are the falling weight Deflectometer (FWD), the deflectograph and the Benkelman beam (deflection or rebound method). Tests are normally conducted in the wheel tracks at intervals of 5 m to 100 m (a minimum of 10 points per uniform section and a preference of more than 30 points) (Fwa, 2006). A more intense frequency of deflection measurements may be desired in areas that appear to have high variability in pavement structure or condition. Measurements between wheel tracks are prudent where the pavement in a less trafficked area may need to be assessed. Measurements on pavements with bituminous layers should be accompanied by temperature measurements.

Deflections can be erratic due to the movement ("rocking") of the slabs during the test on pavements with stabilised base courses. Stiffness of the stabilised layer varies in cases between the wheel path and in between the wheel path. Deflection testing could therefore be staggered to quantify the deterioration of the stabilised layer.

Deflections provide information that is used to calculate layer stiffness as will be discussed in a further section.

4.1.6 Ground Penetrating Radar (GPR)

GPR provides a continuous stream of layer thickness data and can be obtained at relatively high traffic speeds. A drawback to GPR is that it can be expensive for relatively small projects due to the mobilisation cost, and usually requires analysis by highly skilled geophysicists to produce reliable results.

There are two main types of GPR: air-shot and ground-couple. Air-shot measurements are typically performed at highway speeds using a van with one or two transmitter/receivers. The frequency of the waves produced by the transmitter is selected to produce good resolution at different depths in the pavement. High frequency waves produce good resolution near the surface of the pavement, while lower frequency waves produce better resolution deeper in the pavement. The fact that the wave travels through air prior to entering the pavement introduces some uncertainty with air-shot signals compared to ground-couple signals. On the other hand, ground-couple equipment can usually only be used at walking speeds, which makes it difficult to use to characterise pavement layer thicknesses on anything other than very short projects.

Some coring and DCP testing or sampling is required to provide "ground truth" to produce reliable GPR results. Coring, DCP testing and sampling can be performed after the GPR survey and is used to provide baseline layer thickness and identification for calibrating the analysis of the GPR signals, and to resolve questions where the GPR signals show anomalous results.

Collected data is analysed using software to filter out unwanted signals and provide repeatable results. GPR has been reported (Lahouar et al, 2002; Shin and Grivas, 2003; Aperio, 2004; Al-Qadi, 2005) to be effective in providing information of transportation facilities including:

- Pavement layer thicknesses;
- hidden construction changes;
- changes in subgrade conditions;
- position of buried services;
- identification of debonding of bituminous layers;
- relative assessment of the integrity of bituminous pavement layers, and
- voids underneath concrete slabs.

Recent developments have also provided information on moisture condition and density of unbound material layers. If more than one scanner is used during data collection, a three dimensional (3D) image of the pavement structure can be produced.

4.1.7 Spectral Analysis of Surface Waves (SASW)

The use of seismic technology in pavement evaluation in Southern Africa is still in a research and development stage. Currently three Portable Seismic Pavement Analysers (PSPA) are in use in Southern Africa (Steyn and Sadzik, 2007). The PSPA is a non-destructive device used for the evaluation of the seismic stiffness of a pavement structure using the Ultrasonic Surface Wave (USW) method. The seismic stiffness is measured by applying a stress wave to the pavement surface and recording the velocity of the wave through the pavement structure. Factors such as layer thicknesses, material type and material density affect the way in which such waves are reflected and attenuated in the pavement.

The frequency or energy of the applied load to the pavement will determine the depth to which the applied wave will penetrate into the pavement and the thickness of the pavement that can be evaluated using a specific frequency wave. In general, the lower the frequency of the wave, the deeper the layers that can be characterised, and the higher the frequency the shallower the layers that can be characterised. The PSPA uses a source with a relatively high frequency and the response from the upper pavement layers is thus measured. These moduli are low-strain high-strain-rate moduli, and thus numerically larger than those obtained using a traditional deflection measurement, which causes high-strain deformation at lower strain rates.

4.1.8 Test pits, trenches, cores and laboratory testing

Coring and test pitting are costly but essential to obtain the following information:

- Visual condition of the materials in the pavement, e.g. moisture condition, deformation, deterioration of stabilised layer, carbonation.
- Material properties such as grading, plasticity, binder content, in-situ bearing capacities (CBR), etc. Some of these can be used to estimate properties such as stiffness and fatigue characteristics.
- Reaction of the in-situ material with different treatments to assess different rehabilitation design options.

A wide range of laboratory test methods can be used to determine relevant bituminous material properties, i.e. stiffness and fatigue strength (3 or 4 point loading of beams, indirect tensile loading of cores), and pavement deformation characteristics (repeated axle loading, wheel tracking). Resilient properties of cementitious pavement materials can be determined from indirect tensile tests. However, these properties are more commonly inferred from unconfirmed compression strengths (TRH13, 1986). The different tests can give highly variable results due to differences in sample preparation and loading. Results should therefore be used with circumspection.

Fewer laboratory tests have been used to measure the resilience and deformation properties of granular pavement materials. The predominant methods are the tri-axial and the K-mould tests. Results from these tests will only be reasonable simulations of the actual behaviour if the loading, confining and moisture conditions are representative of actual conditions in the pavement.

4.2 Selection of the appropriate tests and frequencies

One of the major challenges in the assessment of material properties for pavement rehabilitation design is the selection of the appropriate test, the test frequency, and the interpretation of the data, if (and it often is the case) the material properties determined from the different tests give different properties.

The previous sections contain information on factors to consider, tests that can be used and the material properties resulting from these tests. There are, unfortunately, no hard-and-fast rules for use in the selection of tests. The following are some practical guidelines:

- The first step should always be to gather as much available information as possible. Typical sources are as-built drawings, discussions with site staff, geological data, and pavement management data such as roughness, riding quality, skid resistance and visual assessments.
- The designer must clearly understand the principles of each test and the resultant properties. Disturbed and undisturbed samples represent different conditions and thus material properties. The same is true for temperature, moisture conditions and loading patterns.
- Layer stiffness is essential for ME designs and can best be obtained by the back-calculation of deflections, obviously with the use of the correct back-calculation procedure. Layer thickness measurements at the same points as deflection tests are essential for producing reasonable stiffness results from back-calculation.
- Stiffness determined from tests such as the DCP and CBR are not good estimates of the effective layer stiffness.
- Test pits, trenches, and/or cores should be part of any investigation, even if limited in extent. This is the only way to visually observe material quality, deformation at depth and moisture conditions. Test pits should be placed at characteristic locations in sections where there is little variability, and also at locations where there is uncertainty as to underlying pavement structure and condition such as at localised failures.
- Laboratory testing of materials is expensive, but useful, when properties (such as stiffness, fatigue characteristics) of new materials have to be obtained.
- Extracting cores from lightly cemented layers can be difficult and in-situ testing (DCP, deflections) is preferred.
- Cores are useful to visually observe the condition of the stabilised layer, while UCS and ITS tests will provide additional structural information.
- Phenol red tests in test pits can be used to assess whether carbonation has taken place.
- DCP tests are not useful on highly stabilised layers.

Table 3 indicates which test method can be used to determine the stiffness of various pavement layers.

Table 3: Determination of layer stiffness

Stiffness	**Calculation/estimation method**
Asphalt	• Master curve as function of time and temperature; characterised by tri-axial dynamic stiffness or by flexural and shear frequency sweep • Back-calculation from deflections
Existing unbound layers	• Tri-axial testing • Back-calculation from deflections
New unbound layers	• Primary reliance on tri-axial testing • Soil classification and simple tests
Existing cemented soils	• Estimate from UCS • Back-calculation
New cemented soils	• Estimate from UCS • Use tri-axial test

- Longitudinal (mainly due to poor construction of joints and swelling of subgrades) and transverse (normally due to temperature movements) cracking is often not structural related and further deflection tests will not provide additional information. DCPs and test pits (as well as moisture conditions) may shed light on the causes of longitudinal (settlement) cracks.
- The shape of the deformation is a good indicator of the origin of the rutting. Figure 3 shows the four typical shapes related to the origins of the deformations. Knowledge of the position of the deformation in the pavement should guide the selection of further tests. As an example, if deformation is primarily in the asphalt surfacing then coring and testing of the asphalt layer would be important, while back-calculated stiffness of the base could indicate to what extent the rutting in the asphalt is exacerbated by poor support. On the other hand, if the deformation comes from the subgrade, it would be important to obtain the properties and moisture conditions of the subgrade by means of DCP testing (note: in the dry condition even the DCP tests may not provide relevant information) and test pitting.

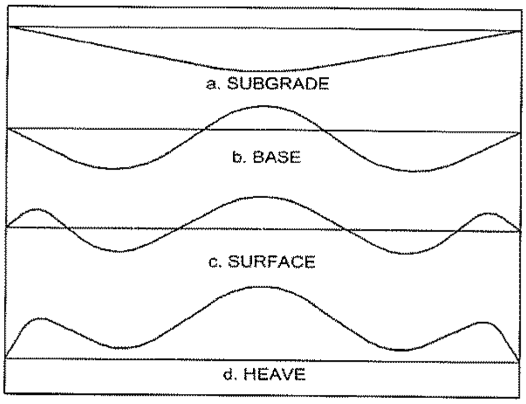

Figure 3: Relationship between shape and the cause of deformation (after Haddock, 2005)

- The season in which deflections are taken must be taken into consideration. High suction during dry periods will increase the back-calculated stiffness of soils. Low temperature will increase the back-calculated stiffness of asphaltic materials, including foamed asphalt and emulsion treated soils.
- Where thick asphalt or asphalt treated layers are present, it is often best to measure at least some of the same locations in the early morning and again in the late afternoon to define the effects of temperature on stiffness.

Last, but by no means least, is the consideration of statistical principles in the selection of frequency of testing (linked to risk profiles) and the cost of testing. Publications such as TRH12 (TRH12, 1997) and Austroads (Austroads 2004b, Austroads 2005a) provide useful guidelines on the statistical selection procedures, while Table 4 lists relative costs of the different tests.

Table 4: Relative cost of assessment methods

Assessment	Relative cost per km
Visual assessment (TMH9 method)	1 unit
Roughness, rutting and macro-texture (continuous per km)	1.5 unit
Skid resistance (micro) (continuous per km)	1 unit
FWD deflections (20 per km)	10 units
DCP (5 per km)	14 units
SASW (per point)	1 unit
GPR (continuous per km)	12 units
Test pit and core (per test)	6 units
Laboratory tests (PI, GM, CBR) per sample	7 units
Core (per core)	5 units

4.3 Interpretation of information

Deflections could be low on pavements where rutting is in the asphalt surfacing, whereas deflections on pavements with deformation of the subgrade are normally high.

The selected tests produce a wide range of information which has to be interpreted and combined to provide a clear picture of the condition of the pavement and realistic material properties. Some of the results, e.g. visual condition, riding quality and rutting, provide information that can be used to optimise further testing and to understand the mode of failure. Examples are:

- The degree and extent of the blocks and stabilisation cracks can give an indication of the amount of stabilisation (large blocks with wider cracks indicate more highly stabilised materials) and the deterioration that has taken place (small blocks, pumping in the wheel paths).
- A correlation between rutting and deflection, i.e. high rutting and low deflection, indicate base and/or sub-base deficiencies. (TRH12, 1997).
- An overstressed subgrade can be inferred from a correlation between rut depth and deflection (an increase in both), or a correlation between riding quality (decreasing) and deflection (increasing). GPR, after correlation with cores, can be used to determine the layer thicknesses (of some layers) on a continuous basis. DCP readings can be used directly in the DCP design or converted to CBR (Jordaan, 1989a; Gabr, 2000) or elastic moduli (De Beer, 1989). Similarly results from laboratory tests can be converted as estimates of layer stiffness and fatigue characteristics (Brown, 1994; Edwards, 2005).

Surface deflections can directly, as normalised maximum deflection, be used to calculate remaining life and required asphalt overlay thickness (AI, 1989; Lister, 1972, TRH12, 1997) or for the calculation of deflection bowl parameters (SARB, 1992). Adjustments for temperature and moisture must ideally be made.

Arguably, the main use of the deflection measurements is the calculation of in-situ, or effective layer stiffness. This is a very important material property and warrants a more detailed discussion.

Since most pavement materials are stress-sensitive, the effective stiffness of pavement layers is influenced by a large number of conditions (see Table 5) e.g.:

Table 5: Factors affecting the stiffness of asphalt pavement materials

Material type	Traffic conditions	Climate conditions	Location considerations (wheel path/centreline)	Other considerations
Granular materials	Traffic stress (load, contact stress)	Recent rainfall (suction) Moisture content Temperature	Compaction by traffic	Construction compaction Confinement (presence of stiff layers above and below)
Clay materials	Traffic stress (load, contact stress)	Moisture content	Compaction by traffic	Construction compaction
Asphalt materials	Traffic loading time (speed) Traffic stress (load, contact stress)	Temperature Moisture content	Compaction by traffic Damage from traffic	Construction compaction Age (exposure to hot temperatures, air)
Cemented materials	If lightly cemented: traffic stress (load, contact stress)	Low temperatures (development of thermal cracking)	Damage from traffic	Age (curing time for hydration of cement; development of shrinkage cracking)

- Thickness and stiffness of overlaying layer and the stiffness of the supporting layer influence the stiffness of a layer. De Bruin and Visser, (De Bruin, et al, 2002) clearly demonstrated that stiffness of a pavement layer changes before and after rehabilitation. The effect of the position of the layer in the pavement is further borne out by different stiffness criteria used for a layer, which is calculated from deflections tested directly on the constructed layer versus that calculated from deflections on an overlaying layer, e.g. sub-base stiffness. For typical granular sub-base layers, which are normally stress stiffening, stiffness can be 400 MPa versus 250 MPa depending on whether the deflection tests are conducted directly on the layer or on the layer above (Austroads 2004a).
- Austroads suggests different stiffnesses depending on the thickness of the overlaying material and stiffness of cover material (Jameson, 2002).
- The moisture condition (the soil suction in particular) (Brown, 2004).
- Temperature and loading frequency, for bituminous materials in particular (Jameson).

Changes in any one of these conditions will change the measured stiffness. Designers should understand the conditions of the test when selecting stiffness values to use for design, and expect major differences in measured stiffness when the conditions are different. The designer should select stiffness values that will be as representative of the stiffness under the expected conditions of the pavement in the field. Newer design methods increasingly consider changes in field conditions when determining stiffness values.

Recent analysis of moduli for several granular materials back-calculated from Multi-Depth Deflectometer (MDD) and Falling Weight Deflectometer (FWD) deflection data on Heavy Vehicle Simulator test sections, showed that the stiffness of granular materials increased as expected under heavier loads, but decreased under constant load when the stiffness of the asphalt concrete surface was reduced by either temperature or damage (Ullidtz et al, 2006).

It has been hypothesised that the changes in the modulus of the granular layers cannot be explained solely by the stress dependence of the material. It may be necessary to consider the particulate nature of the material acting under the confinement provided by the "plate" stiffness of the asphalt concrete, a function of the asphalt concrete stiffness and the upward and outward displacement of particles away from the load. Analysis by Ullidtz (2002) using two-dimensional Distinct Element Method analysis found much greater upward and outward displacement of particles compared to calculations using continuum mechanics. Confinement for use in the equations described above is normally calculated directly under the load using layered elastic theory, an application of continuum mechanics, and the effects of confinement around the load are not considered. It is also hypothesised that stiffness back-calculated from deflections in the pavement in the field may differ from those based on tri-axial testing and layered elastic theory calculation of confinement because of these effects.

Stiffness of in-place pavement layers are most commonly determined from back-calculation from deflections, or a surrogate method of assessment. The overall bending resistance is approximately a function of the stiffness of the layer times the thickness to the third power. Therefore, the understanding of the stiffness of each layer is highly dependent on knowing the thickness of the layer (which can be determined, in cases, by the GPR). Unreasonable stiffness results are often obtained due to variability of the thickness of the layers. If the layer is thinner than is assumed for the back-calculation then its back-calculated stiffness will be unreasonably low; and vice versa if it is actually thicker than assumed.

Only the effect of the load changes is considered in the MEPDG calculations. Confinement is calculated in sub-layers of the base layer.

The MEPDG considers the effects of load changes and resulting confinement on base and subgrade stiffness, and also considers the confining effect of the stiffness of the asphalt layer. Recent analysis of moduli for several granular materials back-calculated from Multi-Depth Deflectometer (MDD) and Falling Weight Deflectometer (FWD) deflection data on Heavy Vehicle Simulator test sections, showed that the stiffness of granular materials increased as expected under heavier loads, but decreased under constant load when the stiffness of the asphalt concrete surface was reduced by either temperature or damage (Ullidtz et al, 2006).

An alternative to highly accurate thickness measurements is to assume that back-calculated stiffness represents the "equivalent stiffness" of that layer, meaning the Eh^3 of the layer, assuming the thickness of the layer. This approach will often result in widely varying and sometimes unreasonable "stiffnesses" back-calculated from the deflections.

Back-calculation requires judgement, experience, an understanding of the limitations of both the measurement and analysis tools available, and good information about the pavement structure. On the other hand, compared to laboratory characterisation of pavement materials, back-calculation offers the following significant advantages:

- Back-calculation provides stiffness of the pavement layers acting in a pavement system, as opposed to a laboratory test of one material with assumed boundary conditions (confining stress, etc.). For this reason, back-calculated stiffness may vary considerably from laboratory results, such as tri-axial test results.
- Back-calculation measures the stiffness of the materials as-constructed, rather than prepared in the laboratory. The density, effects of compaction method (amount of shearing, vibration, etc), densification under traffic, moisture condition (suction, water content) and other effects of field construction are captured.
- The load pulse and stress levels are close to "real".

Major limitations that must be considered with experience and judgement to produce useful results include:

- The constitutive relation assumed for all materials is not correct, and the influence of plasticity, non-linearity, time effects and material and stress state variability, vertically and horizontally within each layer within the deflection bowl, are not considered.
- Bonding conditions between layers are typically not considered in the back-calculation, and less than perfect bonding can result in lower stiffness.
- Definition of the layers in the model of the pavement can change the results. In particular, the back-calculated stiffness of thin and/or softer layers can be unreasonable because of domination of the deflection results by thicker and/or stiffer layers. Great care and application of judgement should be applied when using back-calculated stiffness for thin and/or soft layers. Combination with other layers and consideration of laboratory results are important considerations when developing representative stiffness for these layers. For very thin layers, it is often best to use a fixed reasonable stiffness from laboratory testing or other information, rather than try to back-calculate them.
- It is best if the approach for calculating critical responses in the design method produces response values that are consistent with field measurements of those same responses. The approach used in the design method includes the method of characterising stiffness, the modelling of the interactions of different layers (slip, stress dependencies, etc), characterisation of the traffic load, and the numerical engine that calculates the responses. This objective is constrained by the use of continuum mechanics for particulate materials, the non-elastic and non-linear nature of many materials, and difficulty in characterising bonding and other boundary conditions.
- The method of characterising stiffness when using an ME design procedure must produce critical responses consistent with the stiffness characterisation used for calibration of the design method. If not, the pavement design will vary depending on the stiffness measurement method used. This is an extrapolation of the design method outside of the context in which it was developed, and often will not produce reliable results.
- If reliability is not explicitly considered in the design method, the user of the method must be aware of any reliability built into the method by its developers. If conservative values were incorporated in the calculation of the critical responses or the development of the transfer functions, then further application of reliability, conservative values, or factors of safety may result in an over-designed structure.

It is not uncommon that material properties obtained from the different tests lead to differing (even seemingly conflicting) results. For example, it is unlikely that the different methods, e.g. DCP, maximum deflection AASHTO and ME method will produce the same remaining life and required overlay thickness, or that a DCP calculated CBR of an in-situ layer would be the same as the CBR determined in the laboratory on a disturbed sample.

This creates a dilemma for the designer. And again, there are no hard and fast rules, only some practical suggestions namely:

- Long and Jooste (2007) developed a novel procedure of combining results from different material tests to classify the material in one of the TRH14 (1986) classes. The method provides a procedure to convert test results obtained from a number of tests (e.g. DCP penetration, FWD stiffness, grading, PI, CBR, etc) into a design equivalent (DE) material class. Layer properties (i.e. stiffnesses) and the type of failure / transfer function) can then be estimated and used in the ME analysis procedure or converted to a pavement number (PN), which is correlated to a bearing capacity. Estimated stiffnesses should be used with circumspection when reliable back-calculated or laboratory determined values are available for use in the ME method.
- Layer stiffness obtained from conversion from properties such as DN and CBR should be used in the ME method with great circumspection and not when back-calculated stiffness results are available.
- Back-calculated stiffness must reflect the actual situation (also see section 4.3). For instance, the back-calculated stiffness of the stabilised sub-base should not be low, if it can not be penetrated with a DCP or if a drilled core is intact.
- All determined material properties and predictions must correspond with actual conditions. An analysis that predicts cracking in the asphalt layer as failure, while there are clear signs of existing rutting due to subgrade deformation, has to be questioned.
- Empirical methods, such as maximum deflection, DCP remaining life predictions, and AASHTO must be viewed in the context of the information used to develop the relationships.
- Adjustment for temperatures in asphalt and to some extent bound layers is fairly well-established, but adjustment for moisture (soil suction) in other layers is not that advanced. No or incorrect adjustments can also significantly affect material properties.

In the end, designers must realise that different tests can produce varying results, but describe a different aspect of the material. As long as the determined material properties are used in the appropriate method and within the right context, this should not prevent the designer from producing an optimum design.

5 CONCLUSION

Pavement rehabilitation design and the determination of material properties are complex issues. A holistic approach should be used whereby all available methods are being considered and the appropriate ones used. Different test methods provide different material properties, which all contribute to a better description of the situation.

A realisation and understanding of the design methods and test procedures are essential for the designer to determine material properties for pavement rehabilitation design. The paper provided information on the designs and test methods.

The selection of tests and frequencies and the interpretation of the information should be based on recognising the principles behind the tests, and reflect the actual situation.

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KEYWORDS

Pavement rehabilitation; mechanistic-empirical, material properties, layer stiffness.

APPENDIX E

ROAD PAVEMENT CRACK SEALING: EXPERIENCES IN THE REPUBLIC OF SOUTH AFRICA

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ABSTRACT

There is widespread acknowledgement in the Republic of South Africa that bituminous pavement cracks are formed either from the top down or from the bottom up.

The top open cracks normally are associated with environmental conditions such as heat, cold thermal movements, etc. It is also caused by overloaded tyre contact stresses, which exceed the limiting strain offered by the asphalt materials. These cracks are normally wide open and crack sealing from the top is possible and relatively easy. This paper describes the South African method (materials, machine and procedures) for sealing top down cracks.

The top closed crack caused by fatigue, structural inadequacies etc., are normally witnessed as fine cracks, too narrow to be sealed from the top. Because of the tightness of the bottom up cracks, they are invariably treated by applying a sheet type sealant system. The normally used sealant sheets comprise either a bitumen impregnated geofabric, geogrids, patented sealant sheets, chip and spray constructions or asphalt overlays. In the RSA use is made of chip and sprays to seal the entire width of roads which then acts as a stress absorbing membrane (SAM). This paper describes the seal type selection, seal materials, equipment and methods to seal the bottom up cracks.

1. OVERVIEW

Crack sealing should be designed to serve three basic purposes:

- to prevent the ingress of surface water into the base course and underlying layers;
- to restrain the pumping out of fines and so maintain an uniform support; and
- to serve as a stress absorbing system.

There is widespread acknowledgement in the Republic of South Africa that bituminous pavement cracks are either the top open or the top closed type.

1.1 Crack Activity Measurements

The differential movement of the cracks are measured using the Crack Activity Meter (CAM). The sensors of the CAM are glued down, one on each side of the crack, a vehicle loaded to 80 kN standard axle load is driven over the crack and the sensor records the vertical and horizontal movement across the crack. Table 1 below divides the crack movement in different classes.

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Table 1. Crack activity measurement classification.

Crack movement	Classification
< 0.1 mm	Low
0.1 mm to 0.2 mm	Medium
0.2 mm to 0.3 mm	High
> 0.3 mm	Very high

1.2 Top Open Cracks

The top open cracks normally are associated with environmental conditions such as heat, cold, thermal movements, etc. It is also caused by overloaded tyre contact stresses, which exceed the limiting strain offered by the asphalt materials. These cracks are normally wide open (3mm or more) and crack sealing from the top is possible and relatively easy.

This paper describe the South African method (materials, machine and procedures) for sealing top open cracks which consists of:

- cleaning all cracks with hot air lances
- spraying vegetation killer in cracks where necessary
- deep jetting in of bitumen to prime the crack sides
- selection process to find the appropriate 'main sealant type'
- construction of main seal
- bandage of sealant over crack to serve as stress absorbing membrane (SAMI)

1.3 Top Closed Cracks

The top closed crack caused by fatigue, etc., are normally witnessed as fine cracks, too narrow to be sealed from the top. However, these cracks may widen towards the bottom. Because of the tightness of the top closed cracks, they are invariably treated by applying a sheet type sealant system. The normally used sealant sheets comprise either a bitumen impregnated geofabric, geogrids, patented sealant sheets, chip and spray constructions or asphalt overlays (conventional bitumen, modified bitumens, bitumen rubber seals, etc.). In the RSA use is made of chip and sprays to seal the entire width of roads and serve as stress absorbing membrane (SAM). Much use is also made of the chip and spray seals as a stress absorbing membrane interlayer (SAMI) i.e. where the seals are overlayed with an asphalt. The SAMI not just provide water tightness but also largely reduces the required thickness of the asphalt overlay. The RSA tables for structural equivalent asphalt thicknesses in overlay designs are also furnished in this paper (table 7). This paper describes the laboratory testing, seal type selection procedures, seal materials, equipment and methods to seal the bottom up cracks.

2. TOP OPEN CRACKS (3MM OR WIDER)

The top open cracks are normally associated with environmental conditions such as heat, cold, thermal movement. It is also caused by overloaded tyre contact stresses which exceed the limiting strain of asphalt. These cracks are normally open on the surface and can be sealed from the top.

2.1 Aim

Relative movements between adjacent slabs cannot be prevented through crack sealing but the cracks must remain sealed against water ingress long after reflective cracks break through the new overlay.

To achieve a proper infill it is necessary that the seal:

- is bonded properly to the crack sidewall;
- is soft enough not to impede differential movement across the cracks, thereby obviating the possibility of subjecting the adhesion interfaces to shear forces;
- will return to its original position after movement so that the crack remains sealed for as long as possible; and
- serves as a SAMI (stress absorbing membrane interlayer) if the sealant is plastered over the crack in a neat manner.

For better adhesion it is necessary to clean dirt and asphalt from the crack sidewalls properly and to provide as much bonding area as possible. This requires a deep penetrating prime and seal which, in addition, offers a long hydraulic path to resist water movement. The prime and sealant itself must therefore penetrate readily into small crevices, have good adhesive properties and should remain elastic and pliable for as long as possible. Finally, should moisture enter the underlying base during the terminal phase of the crack seal, then the old sealing material will still play a role in limiting the pumping out of fines. The resulting straining out of fines will largely assist in maintaining pavement support underneath the surfacing layers, and so counteract deformation and failure in the vicinity of the crack.

2.2 Types

The most common crack patterns are:

- Crocodile cracking – normally a traffic induced type of crack with initial longitudinal cracks in the wheel path followed by secondary transverse cracks, all to resemble the pattern on a crocodile's skin. Brittleness and other material deficiencies will accelerate the crack mechanism. The end result is a crazed and cracked area within the wheel path with closely spaced cracks, narrow in width (below 3mm) and very difficult to seal with crack infilling, normally recycling of the layer or a surface seal is applied (refer section 3).
- Block cracking – usually caused by shrinkage of cemented layers. Normally the cracks are far apart (3m or greater spacings) and wide (3mm or wider). These cracks lend themselves to infill sealing because the large width and because a surface seal will cover large sound areas, resulting in a waste of sealant.
- Longitudinal cracking – usually originates from poor construction or problem materials. Initially the cracks are wide open and suitable for a crack infilling type of seal. Exceptionally wide cracks can form when the fills settle and move, leading to cracks much wider than 15mm.
- Transverse cracking – this type of cracking is normally associated with cracks wider than 3mm and suitable for infill crack sealing. Blanket surface seal cover could then be wasteful.

The choice between whether infill cracking or a surface seal is to be constructed depends on practicalities and cost. In general, a visual survey is conducted to determine crack widths using a feeler gauge while the crack activity can be measured using the CAM. It should, however, be noted that crack movement can be temperature and load sensitive. In practice cracks narrower than 2mm are generally not sealed. It is, however, possible to seal them by injecting modified emulsion or MSP 1 prime into the cracks. These crack are best treated by either reconstructing that layer or applying a blanket surface seal to cover the required area. Cost and engineering requirements will dictate which option should be applied.

Cracks wider than 3mm are readily sealed by the infill methods described in this paper. However, when incidence of wide cracking is too high then infill cracking becomes too expensive. In general, when the crack incidence is more than 3 linear metres of cracks per m^2 of pavement, then surface sealing is more economical. In such cases wide open cracks can be blown clean by compressed air prior to surface sealing to let some of the surface spray run down into the cracks.

The cost break-even point between infill crack sealing vs. surface sealing depends on the unit rates and should be calculated for each contract.

2.3 Methodology

2.3.1 Cleaning of cracks

The cracks are blown out with a mobile compressor capable of discharging $3m^3$ compressed air per metre at 650 kPa pressure. The compressed air is heated by a heat exchange apparatus so that compressed air is used to blow out and clean the crack. The compressed air is heated using liquid petroleum gas ignited by a spark plug. The temperature of the hot air shall be adjustable up to 300°C. The main advantage of hot air blasting is that all of the old and aged asphalt on the crack sidewalls is removed, thus leaving a fresh face for the prime and sealant to adhere to.

2.3.2 Vegetation killer

Weed killer, dissolved in water should be sprayed into all cracks with vegetation using rucksack pressure type spray cans. Shoulder cracks harbour more vegetation and deserve special attention. A total herbicide, 'Hyvar X', as weed killer two days before cleaning the cracks is recommended.

2.3.3 Priming of cracks

Prime should be jetted into the open cracks using compressed air propulsion. An inverted prime, such as MSP 1, is recommended. A schematic drawing of the equipment is shown in Figure 1. Essentially, the process consists of a blowpipe with a nozzle to direct the jet of compressed air mixed with prime into the crack. A venturi device is fitted inside the blowpipe to create a pressure differential for sucking in prime from a storage vessel.

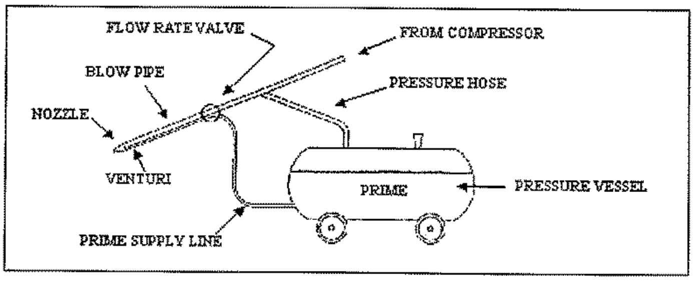

Figure 1. Prime injector.

A closed pressure vessel prime tank, where the air space above the prime was pressurised using a tap-off from the same compressed air source, can be used. This arrangement generates similar head losses across the venturi as across the prime, and with the prime at a slightly higher pressure than the downstream end of the venturi, a constant rate of prime feed can be obtained. A needle and seed valve can be installed in the prime line for the fine adjustment of the air to prime ratio. A complete shut-off valve should be added to isolate the injector from the compressed air. This is most important when public traffic occupies the same carriageway, as the prime injectors have a considerable 'shooting range' of up to 10m.

When jetting the cracks with prime, the sides of the cracks must be thoroughly wetted but without flooding the cracks. The prime must then be allowed to soak and dry before the first application of crack sealant is applied. The prime largely assists in creating a proper bond between the crack sidewall and sealant.

2.3.4 Sealants

Three types of sealant can be used:

- warm bitumen-rubber sealant or cold emulsion sealant (main seal)
- final seal (rubber crumb slurry)
- wide crack infills

Warm bitumen rubber sealant:
The bitumen rubber is heated in a crack sealer tanker to the specified temperature and circulated continuously. A hand-operated lance is used to inject the bitumen-rubber into the crack by pump force. Directly after the crack has been sealed a scraper is used to remove surplus bitumen rubber from the surface. The workmen get trained to strike the said bitumen almost level with the surface so that this bandage can act as a SAMI.

The bitumen rubber consisted of the following mix by volume:

- Bitumen (60/70 penetration grade) 76%
- Rubber (finely ground) 22%
- Extender oil 2%

Table 2. Properties of bitumen rubber.

Property	Min	Max	Test method
Ring and ball softening point (°C)	65	80	ASTM D36
Compression recovery: 5 min 1 day 4 days	 85 70 25	 100 90 55	BR3T
Resilience	10%	35%	BR2T
Flow	0mm	70mm	BR4T

Other commercial types of warm sealant can be used (e.g. Viaflex) which is a highly elastic, hot applied penetration grade bitumen, modified with a high percentage of an electrometric polymer. This product must be applied at a temperature of 130°C with an applicator or pour directly into the cracks with a small container.

Table 3. Properties of Viaflex.

Property	Test value	Test method
Ring and ball softening point (°C)	80 min	ASTM D36
Elastic recovery @ 10°C (%)	80 min	ASTM D113 (mod)
Flow at 60°C (mm)	0	BR4T
Ductility @ 10°C (mm)	500	ASTM D113

Final seal (Rubber crumb slurry):
A slurry seal, using rubber crumbs instead of aggregate can be used to top up and bandage over the wider active cracks. The slurry then serves as a SAMI. Hand tools are used to mix and apply this slurry seal.

The slurry consisted of the following mix by volume:

- Rubber crumbs 60%
- Stable grade bitumen emulsion35%
- Revertex 3%
- Cement2%

Also acceptable is that the previously described warm bitumen rubber sealant (or alternative) be struck off with the surface leaving a bandage thicker than 3mm and 50mm wide. This bandage will also serve as a stress absorbing membrane interlayer.

A cold applied crack sealant (e.g. Petroseal) can also be used as a standby product as it is not cost-effective to keep a specialised machine, such as for hot sealants, on site for the full duration of the contract. The cold-applied crack sealant is of a medium thick consistency which makes it pourable and spreadable.

The cold sealant consists of a rubber polymer and filler which imparts toughness and elastic properties to the product. Undiluted the product sets within an hour or two depending on weather conditions. Cracks wider than 3mm can be sealed by applying the product through a purpose designed applicator with suitable nozzle or pouring the product into the crack and spreading the excess by squeegee. Fine cracks may be sealed by spreading the product over the cracked area with squeegees and finally gritting the area (if required).

The cold-applied crack sealant can also be mixed on site by using an anionic emulsion and latex. The latex must contain 65% styrene butadiene rubber. The sealant (Petroseal) had the following characteristics:

Table 4. Properties of Petroseal.

Property	Min	Max	Test method
Ring and ball softening point (°C)	53	63	ASTM D36
Penetration value 25 °C 100g (0.1mm)	55	75	ASTM D5
Elasticity @ 10°C	1 000	-	ASTM D5
Recovery (%)	50	-	ASTM D113

Wide cracks:
Cracks wider than 15mm, such as when clays shrink are too expensive to fill with infill materials. A mixture of 1 part washed river sand, plus 1 part slaked lime and water to make a wet slurry must be poured into the crack until 20mm below the surrounding surface. Allow a period for this mixture to settle and dry, refill if necessary, and then prime using MSP 1 or equivalent and fill to the level of the surrounding surface using a rubber crumb/crusher dust slurry.

3. TOP CLOSED CRACKS (3MM OR NARROWER)

The top closed cracks caused by fatique etc. are normally witnessed as fine cracks and numerous, it represents a crazed area. Because of the tightness of these cracks and the many linear metres of cracks in these crazed areas are they invariable treated by applying a sheet type surface sealant (or the road be reworked if too severe).

3.1 Aim

Surface seals such as bitumen rubber single seals and geotextile fabrics are prime examples of this group. Normally applied as a surface dressing covering a wide area, if not the total road width.

A proper surface seal should provide for:

- prevention of surface water ingress
- reduce the stress on the crack tip (SAMI)
- bridge the crack opening
- retard reflective cracking
- tolerate large deflections
- stabilise pavement moisture content
- prolong fatigue life

There have been a number of attempts to examine the problem of minimising reflection cracking in asphalt overlays. One such study has been concerned with the influence of rubber-bitumen interlayers (SAMI's). In this study the finite element procedure was used to examine the distribution of stress in an overlay in the vicinity of a crack for situations with and without the SAMI at the interface. Figures 2 and 3 illustrate the effective stress distribution for a specific situation with and without the interlayer. Comparing these figures it is apparent that there is a large reduction in stress levels when the crack sealing is finished off in such a way that it acts as a SAMI underneath the proposed layer.

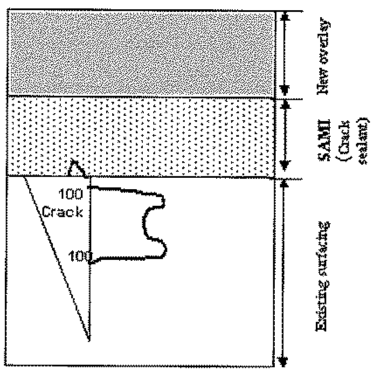

Figure 2. Effective stress distribution in asphalt overlay (in psi) with stress absorbing membrane interlayers (SAMI).

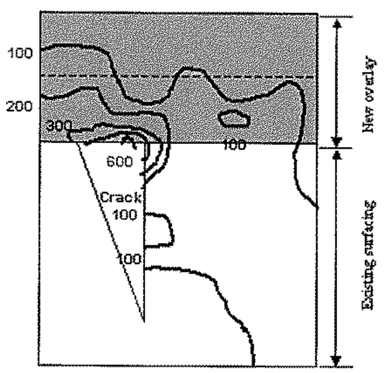

Figure 3. Effective stress distribution in asphalt overlay (in psi) without stress absorbing membrane interlayers (SAMI).

3.2 Geotextile Fabrics

The use of geotextiles (e.g. Sealmac) has become standard practice over the last couple of years. This system is easily applied by hand with a team of say 5 workers which can lay 2km of seal in a day.

The method consists of properly cleaning and sweeping the road to be geotextile sealed. Generally the strips are 200mm wide but full sheets of 2,1m is also available. Where discreet cracks are to be covered, sheets measuring 0.5-1.0m wide are used to seal areas of cracking or potholes. A typical specification for the geotextile appears in table 4. The geotextiles are glued to the road using a tack coat such as cationic 60 or anionic 60-grade emulsion.

An average spray rate of 0.8ℓ/m^2 is used, the geotextile is rolled out onto the wet tack coat by hand or machine and rolled with a pneumatic tyred roller. A saturation coat of the same emulsion is then hand-sprayed thereafter. The application rate being normally around 1.2ℓ/m^2 to render a total emulsion application of around 2.0ℓ/m^2. In large area application where bitumen distributors are justified, hot 150/200 penetration bitumen can be used in the tack coat and saturation layer. Thereafter a medium grade crusher dust is applied to the wet system to render a wearing course with average thickness of say 5mm. The road can then be opened for traffic or, alternatively, it can be rolled further with a pneumatic tyred roller. The bandage can then be sealed using a conventional chip and spray.

Table 5. Typical geofabric for crack bandage.

Property	Requirements	Sealmac	Test method
Tensile strength (kN/m)	7.0 min	8.8	ASTM D4632
Elongation at break (%)	50% min	55%	ASTM D4632
Bitumen retention (ℓ/m^2)	1.1 min	1.8	Task force method 8
Melting point (°C)	150 min	260	ASTM D276

3.3 Bitumen Seals

Normally a modified bitumen chip and spray will be used for this application. Modified bitumens such as SBR, SBS, Bitumen rubber, are used most frequently. In the RSA the bitumen rubber modified binder are used most often (refer table 8a, b, c and d). SBS and SBR modifiers are also used but not as frequent (table 9). The grading for surfacing aggregate is furnished in table 10. The construction of such seals is common practice and a detailed description is not repeated in this paper.

However, some notes on the construction of SAMI's and SAM's follow:

- Normally 19.0mm, 13.2mm or 9,5mm aggregate is used. Of these, 13.2mm is the most popular. An average least dimension (ALD) of respectively 12,0mm; 8.0mm and 6,0mm is required to prevent flaky aggregate so that high binder application rates for crack suppression are possible.
- The aggregate is normally pre-coated to enhance adhesion.
- A high quality chip spreader is required. The spread rate must be checked using 500mm wide canvas strips. The application rate per individual canvas patch should not vary by more than 5% of the average rate.
- The binders are normally very viscous, not self-levelling and leaving tramlining and roping immediately after a spray. Delay chipping for a minute or two to allow the thick binder to flow below the meniscus.
- The spray rate must be tested using a geotextile sheet nailed to the surface and spraying over with the distributor. The geofabric is then cut into 500mm wide strips, weighed and the mass of bitumen calculated for each strip. The spray rate per individual strip should not vary by more than 5% of the average rate.
- When a SAMI is constructed, the authors prefer to have the voids filled up to 80% for suppressing crack propagation. We use a multiplication factor of x 1.6 to convert from a spray rate of 50% to 80% of voids filled. A high softening point is preferred to prevent pick up by wheels when the voids are filled so high up to the top of surfacing.
- When a SAM is constructed and not overlaid for many years, the punching of the chips can give rise to bleeding. Ball pen. tests to assess punching depths are then done and the spray rate adjusted downward. However, a spray rate below 1.8ℓ/m^2 is considered too low to resist reflective cracking for a 13.2mm chip and spray.

- The initial classification of crack movement and the application of modified binders are furnished in table 6.
- The B-R research studies and field surveys revealed that the B-R asphalt, used in conjunction with SAMI in numerous cases, provided superior structural capacity. The structural equivalents are furnished in table 7 below.

4. ANNEXURE OF TABLES

Table 6. CAM classification and treatment.

Crack movement	Classification low	Suggested remedial treatment
< 0.1mm	Low	Conventional surface treatment
0.1 mm to 0.2 mm	Medium	Surface treatment with homogenous modified binder
0.2 mm to 0.3 mm	High	Surface treatment with bitumen rubber
> 0.3 mm	Very High	Thick overlay (e.g. SAMI)

Table 7. Structural equivalents.

Conventional dense graded asphalt (mm)	Equivalent bitumen rubber gap-graded asphalt (mm)	Equivalent bitumen rubber gap-graded asphalt on SAMI (mm)
37	30	-
60	30	-
75	45	30
90	45	30
105	60	45
120	65	45
135	45 BR + 45 (AC)	60
150	45 BR + 60 (AC)	60

Table 8(a). Bitumen rubber: Rubber crumbs specification.

Sieve analysis		Test method
Sieve size (mm)	Percentage passing by mass	
1.18 0.60 0.075	100 (min) 40-70 5 (max)	BR6T (Sabita)
Other requirements		
Natural rubber hydro-carbon content	30% (min)	BS 903 Parts B11 and B12
Fibre length	6mm (max)	
Relative density (gm/cm^3)	1.10-1.25	BR9T

Table 8(b). Bitumen rubber: Extender oils specification.

Property	Requirements
Flash point	180°C (min)
Percentage by mass of saturated hydrocarbons	25% (max)
Percentage by mass of aromatic-unsaturated hydrocarbons	55% (max)

Table 8(c). Bitumen rubber blend specification.

Property	Requirements	Control measures
% of rubber by mass of total blend	20% - 24%	As specified by the supplier
% of extender oil by mass of total blend	6% (max)	
% of diluent by mass of total blend	7% (max)	
Blending/Reaction temperature	170°C – 210°C	
Reaction time (commences when all the rubber crumbs have been added to the blend)	0.5 – 4.0 hours	

Table 8(d). Bitumen rubber binder specification.

Property	Requirements	Test method
Compression recovery: after 5 minutes after 1 hour after 4 hours	 70% (min) 70% (min) 48% - 55% (min)	BR3T (Sabita)
Ring and ball softening point	55 °C (min)	ASTM method D36
Resilience (%)	13% - 35% (min)	BR2T (Sabita)
Dynamic viscosity (Haake at 190 °C)	20-35 dPa.s	BR5T (Sabita)
Flow	20mm – 75mm	BR4T (Sabita)

Table 9. Hot applied modified binders.

Generic type of modified binder		Required properties							
		Grade of base bitumen	Min. softening point (°C)	Min. dynamic viscosity at 135°C (Pa.s)	Min. Ductility at 10°C (mm)	Min. elastic recovery (ductilo-meter) at 10°C	Max stability difference (ring and ball) (°C)	Min. adhesion at 5°C (%)	Min. adhesion at 50°C (%)
Plastometer polymer (EVA)		B4	48	0.5	300	45	2	90	
Elastomer polymer	SBR	B8	47	1.0	1000	55	2	90	100
	SBS	B8	49	1.0	500	60	2	90	
	SBR	B4	45	0.5	1000	55	2	90	
	SBS	B4	47	0.5	500	60	2	90	100
Test method	-	-	ASTM D36	ASTM D4402	DIN 52013	DIN 52013		*Mod Vialit method	

* The Modified Vialit method: See 'Technical Guidelines for Seals using homogenous modified Binders', Sabita Manual 15, May 1994.

Table 10. Single sized crushed aggregate.

Sieve size (mm)	Grade	% by mass passing						
		26.5mm nominal size	19.0mm nominal size	13.2mm nominal size	9.5mm nominal size	6.7mm nominal size	4.75mm nominal size	2.36mm nominal size
37.5	Grades 1 & 2	100	-	-	-	-	-	-
26.5		85-100	100	-	-	-	-	-
19.0		0-30	85-100	100	-	-	-	-
13.2		0-5	0-30	85-100	100	-	-	-
9.5		-	0-5	0-30	85-100	100	-	-
6.7		-	-	0-5	0-30	85-100	100	-
4.75		-	-	-	0-5	0-30	85-100	100
3.35		-	-	-	-	-	0-30	-
2.36		-	-	-	-	0-5	0-5	0-100

5. ACKNOWLEDGEMENTS

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APPENDIX F

SURFACE ENRICHMENT SPRAYS AS A COST EFFECTIVE SOLUTION FOR PREVENTIVE MAINTENANCE

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ABSTRACT

More than 80% of all surfaced roads in southern Africa are surfaced and/or resurfaced with surfacing seals which include sand seals, single chip seals, double chip seals, slurry seals and combination seals e.g. Cape seals.

The application of enrichment sprays has been practiced for more than thirty years and has generally been accepted by major road authorities as a low risk and cheap solution to extend the life of existing surfacings. However, several concerns have been raised in recent years regarding the appropriateness of this preventive treatment and the effectiveness of the various products available.

Following several investigations into problems experienced and arguments related to enrichment sprays, the need was identified to obtain the opinions of practitioners and binder suppliers in an effort to record the current best practice in southern Africa.

1. INTRODUCTION

Diluted emulsion application as enrichment sprays has been practiced for more than thirty years and has generally been accepted as a low risk and cheap solution to extend the life of existing surfacings.

Following several investigations into problems experienced and arguments related to diluted emulsions, the need was identified to obtain the opinions of practitioners and binder suppliers in an effort to publish the current best practice in South Africa as part of the national guideline document TRH3: Design and Construction of Surfacing Seals for the Urban and Rural Environment (1).

The purpose of this paper is to:

- Discuss the purpose of enrichment and rejuvenation sprays.
- Provide some background to the intrinsic properties of bitumen emulsions and inverted cutback bitumen emulsions.
- Discuss the factors causing surface deterioration.
- Provide recommendations for the selection of the appropriate type of enrichment and rejuvenator sprays and emulsion composition
- Discuss the required binder application rates
- Discuss appropriate timing of application
- Provide guidelines for on-site application

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- Discuss the cost effectiveness of enrichment and rejuvenator sprays
- Highlight typical problems and solutions

The paper is structured to consecutively address each of the abovementioned aspects.

The paper could be of value to road authorities, consultants and contractors tasked with decision making, design and application of diluted emulsions on surfacing seals

2. PURPOSE OF ENRICHMENT AND REJUVENATION SPRAYS

2.1 General

Bitumen emulsions can be diluted with water for the following purposes:

- As a cover spray for newly constructed single or double seals to prevent/ reduce aggregate loss
- Enrichment and rejuvenation of dry/porous surfacings, often as a pre-treatment before resurfacing operations
- As a prime coat on granular base courses
- As a tack coat for asphalt overlay
- Slushing of natural gravel base layer on low volume roads prior to surfacing

For the purpose of this paper only the first two applications are discussed in detail.

2.2 Prevent/Reduce Aggregate Loss on New Seals

Aggregate loss sometimes occurs soon after construction due to various reasons such as:

- Too low binder application rates
- Insufficient rolling
- Poor binder/aggregate adhesion
- Cold temperatures
- Open to traffic too soon and traffic speed limit not controlled

One of the most cost-effective ways to prevent premature ravelling could be to add binder to the newly constructed seal in the form of a diluted emulsion.

Due to the fairly coarse texture of the new seal and the need to create a bond between the aggregate as shown in Figure 1, a rapid setting grade emulsion is considered appropriate for this situation.

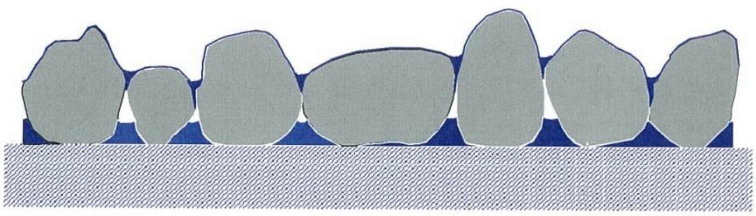

Figure 1. Dilute emulsion cover spray.

2.3 Enrichment and Rejuvenation of Dry/Porous Surfacings

2.3.1 Enrichment sprays

The main function of a diluted bitumen emulsion is to retard stone loss and partly enrich the binder in aged stone seals. Dilution of bitumen emulsion with water lowers the viscosity permitting easier penetration into the surface voids. Refer to Figure 2. The water evaporates leaving a residual deposit of bitumen to improve stone retention. Excess bitumen deposited on the stone is ridden off with traffic.

The most common feeling with enrichment sprays is to get as much as possible new binder as low as possible in the existing seal.

Experience (several case studies) indicate that diluted emulsion application:

- Partly enriches the existing binder
- Drastically reduces permeability of the existing surfacing
- Improves the performance of follow-up resurfacings (slurry seals and stone seals)

Dependent on the initial binder application, enrichment sprays could be applied up to three times during the life of a single 13,2mm seal, each time extending the effective life of the seal with up to three years.

With reference to the above, the appropriate type of binder would be a low viscosity stablemix grade bitumen emulsion that could flow around the existing particles, penetrate and fill up from the bottom. Refer to Figure 2.

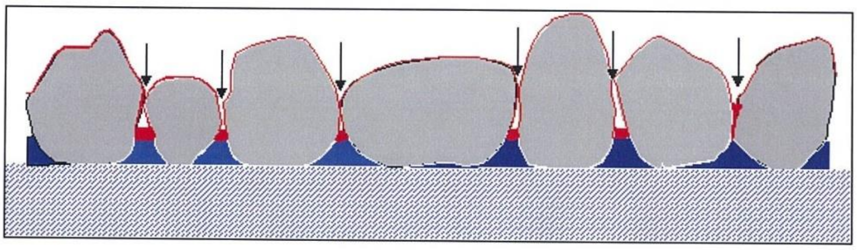

Figure 2. Dilute emulsion enrichment spray.

2.3.2 Rejuvenation sprays

Bituminous surfacings deteriorate with time due to various reasons. In the drier parts of southern Africa and especially on low volume roads, the effect of oxidation and hardening of the bituminous binder result in porous surfacings, surfacing cracks and/ or aggregate loss.

The purpose of rejuvenation is to improve the flexibility of the binder by the addition of aromatic oils and paraffinic cutters, which are lost with time. Standard grade bitumen emulsions do not have the solvency power to penetrate existing dry and brittle surfacings and therefore special inverted cutback emulsions are formulated for this purpose.

Whilst these special emulsions also add new bitumen to the surfacing, they have the added benefit of rejuvenating the aged bitumen. This is achieved via the cutter medium which also helps the rejuvenator penetrate the existing surface up to 10 mm, depending on the type of surface.

Other benefits of rejuvenator sprays are:

- Help close up hairline cracks under traffic
- Prevent stone loss
- Improve surface impermeability

3. INTRINSIC PROPERTIES OF BITUMEN EMULSIONS

3.1 General

The purpose of this section is to provide some background to practitioners regarding the development of different types of bitumen emulsions. A clearer understanding of the purpose of different products should minimize inappropriate specification.

The manufacturing of bitumen emulsions have developed in South Africa over the past 75 years with the aim to provide appropriate products for specific situations and applications.

3.2 Emulsification of Bitumen

Bitumen emulsion consists of three basic ingredients: bitumen, water and emulsifying agents. When bitumen and water containing an emulsifier are introduced into a colloid mill, which is a high-speed dispersing apparatus, the bitumen is broken down into microscopically sized spheres. The emulsifier molecules are adsorbed on the surface of the bitumen particles, lowering the interfacial surface tension between the bitumen and the water, and allow a stable dispersion of bitumen in water to be formed. If the emulsifier imparts a negative charge to the bitumen particles, the emulsion is classified as an anionic emulsion. Conversely, if the emulsifier imparts a positive charge to bitumen, the emulsion will be classified as cationic.

3.3 Emulsifiers

The purpose of the emulsifier is to suspend the bitumen particles in the water phase and to impart specific breaking characteristics to the bitumen emulsion.

Anionic emulsions are prepared with alkaline solutions of long chain fatty and rosin acids, whilst cationic emulsions are prepared with acidified solutions of long chain amines and its derivatives.

The type and quantity of emulsifier used will determine whether the emulsions will have rapid setting, medium setting or slow setting characteristics.

The use of anionic spray and premix grade emulsions have decreased considerably over the last twenty years, as the equivalent cationic grades are more rapid setting due to the chemical interaction with the aggregate. Anionic stable grade emulsions are still widely used.

3.4 Addition of Flux

Illuminating paraffin flux and other petroleum solvents are commonly added to cationic spray grade and cationic premix grade emulsions. The addition of solvent flux to spray grade emulsions enhances the breaking characteristics of the emulsion and chip retention during low temperature conditions. The quantity of flux is varied seasonally and could vary from 0% in summer to 5% by mass of the binder during winter. When solvent fluxes are added to premix grade emulsions, it enhances the coating of the aggregate by the binder and also extends the stockpile life of the cold mixes.

3.5 Product Ranges and Purpose of Development

3.5.1 Unmodified emulsions

Rapid setting emulsions were developed for chip and spray applications. These emulsions are reasonably unstable, and are formulated to have sufficient stability to allow for heating, pumping and application on the road, but after application, the emulsion rapidly breaks down on contact with the aggregate and during mechanical rolling. Surfaces can thus be opened to traffic shortly after application of the aggregate.

Medium setting emulsions have slightly higher stability than rapid setting emulsions. These emulsions are capable of mixing with clean, dust free aggregate and are commonly used for the preparation of cold mixes for patching.

Slow setting emulsions can mix with cement and very fine aggregate particles, allowing for the preparation of slurry mixes and dense graded cold mixes. They are also commonly used for soil stabilisation purposes.

3.5.2 Polymer modified emulsions

Anionic emulsions were first modified with natural and synthetic latex. The setting characteristics of these emulsions were, however, very slow. Cationic polymer modified emulsions are today widely used due to the enhanced breaking behaviour of these emulsions.

Two types of cationic polymer modified emulsions are commonly available in South Africa. These are: spray grade and Microsurfacing emulsions.

These emulsions are normally modified with SBR latex. Modified spray grade emulsions are used for resealing of cracked roads, whilst the Microsurfacing emulsions are used for preparation of rapid setting slurries. The residual SBR content of these emulsions varies from 3 to 5% by mass of binder.

3.5.3 Dilute emulsions

The following grades of bitumen emulsion can be diluted with water and used for enrichment sprays or fog sprays:

- Anionic or Cationic Stablemix 60%
- Unfluxed Cationic Spray 60, 65 or 70%
- Unfluxed Cationic Spray 65 or 70% modified with SBR latex
- Latex modified cationic microsurfacing emulsion

The decision on which type of diluted emulsion to use will depend largely on the purpose and most desirable flow characteristic of the emulsion.

3.5.4 Inverted cutback emulsions

An inverted emulsion is manufactured from a cutback bitumen with the addition of aromatic oil and a cutback medium. Inverted emulsion differs from that of a normal bitumen emulsion in that the water is dispersed in the binder which is the continuous phase. Once sprayed, it penetrates into dried and aged bituminous surfaces, and rejuvenates the binder, thus extending the time required before resurfacing.

When more binder is required in the surfacing it is recommended that bitumen emulsions be used, whilst inverted cutback emulsions are used when the oxidised binder needs to be rejuvenated.

4. SURFACING DETERIORATION

4.1 General

The typical deterioration of a bituminous surfacing is shown in Figure 3.

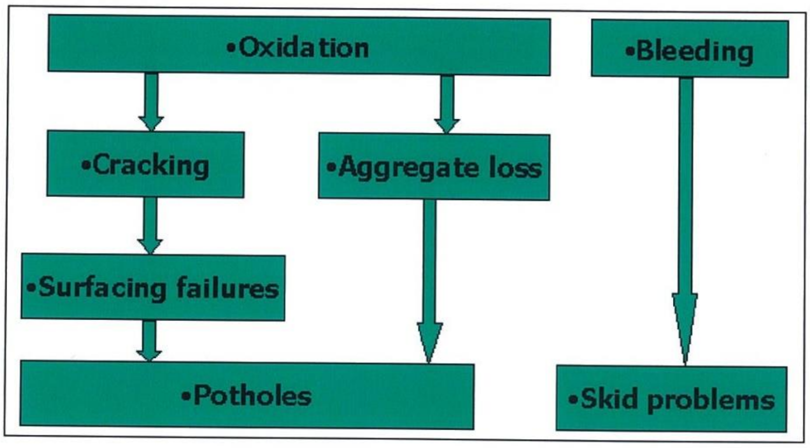

Figure 3. Surfacing deterioration.

4.2 Rate of Oxidation/ Hardening

No formal research results are available to quantify the rate of binder hardening in South Africa. However, based on and opinions of South African practitioners, the hardening of bituminous binders is a function of:

- Exposure
- Film thickness
- Humidity
- Binder quality
- Ultra violet radiation

The maximum air temperatures in all areas of southern Africa exceed 35°C whilst the duration of sunshine varies from more than 80% of possible duration in the north-western parts to 70% over the remainder of the interior to less than 60% in the coastal areas. The high levels of ultra violet radiation coupled with the high summer temperatures are believed to correlate with the rapid aging of bituminous surfaces in the more arid north western parts of southern Africa. Figure 4 shows three different climatic regions based on the so-called Weinert N values (2) (Based on evaporation).

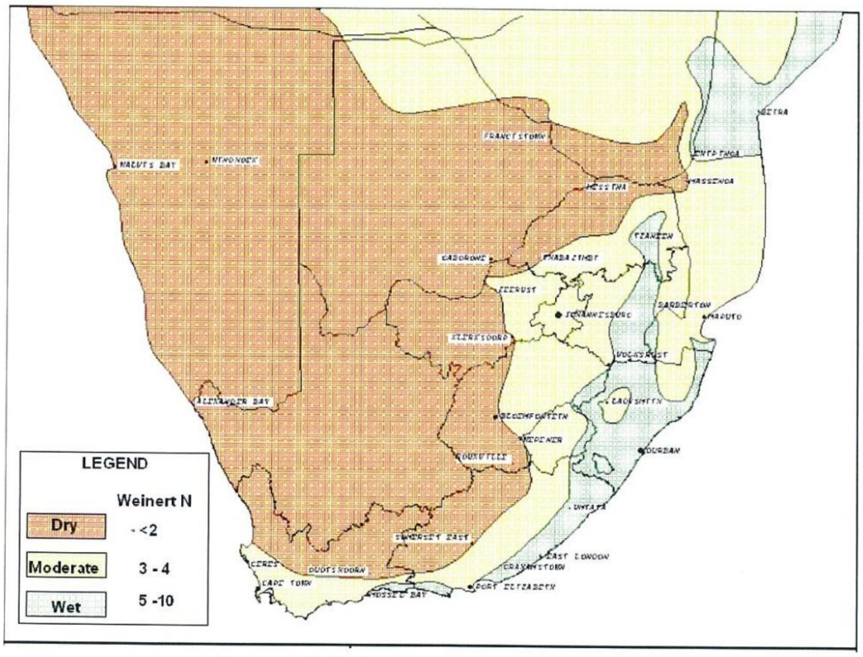

Figure 4. Climatic regions of Southern Africa.

5. SELECTION OF APPROPRIATE MEASURES

5.1 General

The purpose of this section is to assist practitioners in the identification of appropriate situations for the application of diluted emulsions for enrichment sprays.

Refer to TRH3 (1) for initial selection

5.2 Assessment

The purpose is to identify whether

- Additional binder is required to prevent stripping
- The product can and/or should penetrate into the existing surfacing

Constraints influencing the selection are:

- Sufficient voids and or texture exist to accommodate the additional binder
- Gradient of the terrain
- Whether traffic can be accommodated
- Climatic conditions at time of application

5.3 Selection Criteria for Emulsions

Table 1 provides guidelines for selection of suitable emulsions.

Table 1. Selection criteria for emulsions.

Condition	Type of emulsion			
	Stablemix	Spray grade	Latex modified	Inverted e.g.MSP 3
Coarse textured	1	2	2	1
Dense textured	1	2	0	0
Flat gradient < 3%	1	2	1	1
Steep gradient > 3%	0	1 see 6.1	2	0
Dirty surface	2	0	0	0
Cold temperature	2	1	2	0
Hot temperatures	2	2	0	1
Traffic accommodation	2	1	2	0

1 = primary recommendation 2 = secondary recommendation 0 = not suitable

5.4 Selection of Appropriate Composition and Application Rate

5.4.1 General

The purpose of this section is to give some guidance as to appropriate dilution ratios and application rates for different situations.

The factors mostly affecting the performance of emulsions as an enrichment spray are:

- Type and grade of emulsion
- Weather conditions
- Presence of cutters

5.4.2 Type of grade of emulsion

Cationic spray grades will tend to break quicker than anionic and cationic stablemix emulsions. Thus with spray grade emulsions the binder will tend to be deposited on top of the stone chips whereas stablemix emulsions will tend to flow into the voids more easily. Cationic emulsions will render improved binder/aggregate adhesion due to the chemical reaction which takes place between the positively charged bitumen droplets in the emulsion (and latex if used) and the free negatively charged ions of the aggregate.

5.4.3 Weather conditions

The prevailing weather conditions will affect the 'breaking' of a diluted emulsion. In the case of colder weather and in shaded areas, the lower temperature retards the breaking action of the emulsion. In the latter instances quicker breaking Cationic spray grades should be used.

During hot weather, latex modified emulsion will tend to form a skin which could create a false perception that the emulsion has broken. The emulsion must be checked by scratching with a knife to ensure that it is uniformly black under the skin.

In hot weather, the residual binder film tends to become tacky resulting in pick up on the tyres. When the expected road surface temperature is above 50 °C then a higher softening point binder such as 60/70 penetration base bitumen must be considered.

As a general guideline the following closure times should be observed after spraying, based on prevailing weather conditions:

- Hot, windy day: < 2 hours
- Overcast, cool day: < 4 hours

If the road is opened too early to traffic, the fog spray will pick up. If it becomes necessary to open to traffic, use coarse sand to blind areas which are not properly dry in order to reduce the tackiness.

There are some time and drying constraints to be considered when selecting candidate pavements for treatment with inverted emulsions e.g. MSP 3™, particularly in cool conditions or climates with high humidity. When applied on coarse surfaces such as chip seals, traffic can be allowed onto the surface within 4 hours, whilst the drying/penetration time on dense asphalt surfaces could be up to 36 hours.

5.4.4 Presence of cutter

The presence of cutter in emulsion is not desirable when used as an enrichment spray, as the residual binder tends to remain tacky after breaking. It must also be noted that all standard spray grades of emulsion are formulated with between 2% and a maximum of 5% cutter to improve the cohesion development of the residual binder when applied in stone seals. Therefore, when ordering spray grade emulsions for fog spraying purposes, it is important that the binder supplier be requested to omit the cutter in the emulsion formulation.

6. DESIGN

6.1 Binder Application Rates

The texture of the existing surfacing largely determines the application rate. A fine texture requires a light fog spray application whereas a coarse or open textured surface can take a heavier application. An indication of the maximum application rate that a surface can tolerate is definite signs of run off from the existing road surface. The maximum application rate could be up to 1.2 litres per square metre.

The lowest application rate is a function of:

- The minimum spray rate that the binder distributor can spray accurately, which is typically about 0.5 litres per square metre. This equates to a net residual binder of 0.15 litres per square metre for a 50:50 diluted 60% solids emulsion.
- Flow characteristics of the emulsion. Several practitioners recommend a minimum application rate of 0.8 litres per square metre on 13.2 mm seals to ensure that the aggregate is properly covered.
- Some guidelines appear in Table 2 below, but the application rate should be adjusted on-site to avoid problems with over application. In this regard the reader is also referred to the Gautrans manual (3).

Table 2. Application rates.

Existing surfacing	Diluted emulsion (30%) Litre/m²		Inverted emulsion Litre/m²	
	Diluted emulsion	Residual binder	Inverted emulsion	Residual binder
13 mm single seal	0.9 –1.2	0.27 - 0.36	0.6	0.27
13/6 mm double seal	0.8 – 1.0	0.24 – 0.30	0.5	0.23
Cape seal	0.5 – 0.7*	0.15 – 0.21	0.5*	0.23
Asphalt	0.5 – 0.6*	0.15 – 0.18	0.5*	0.23

*Only if surface is open textured

6.2 Binder Content

The bitumen emulsion is normally diluted in a 1:1 ratio with water. This ratio can be varied to overcome certain constraints e.g.:

- In the event of steep grades it is recommended that a spray grade be used with a lower dilution of 70:30 to prevent runoff.
- If the emulsion has to be transported long distances from the source then consideration must be given to using a higher binder content emulsion such as a cationic spray grade 70 or to rather dilute on-site to reduce the effective transport costs.

The lowest recommended binder content is 25% m/m of the diluted emulsion. Diluting the emulsion further will weaken the electrochemical charges thus rendering the end product unstable.

7. APPROPRIATE TIMING

The appropriate timing of diluted emulsion application as a preventive treatment is still considered by many practitioners as an art. The need is often based only on the amount of binder holding the stone and the risk of aggregate loss.

Parameters included in South African assessment methodology to identify the need for seal rejuvenation/ application of diluted emulsions are:

- Macro texture
- Voids to accommodate additional binder
- Dry/ brittleness of the binder
- Aggregate loss
- Low degree surfacing cracking due to hardening of the binder

The appropriate time during the year for application is dependent on the purpose and temperature constraints. Cognisance should be taken that:

- Fine surfacing cracking could close up during summer
- Surface temperatures could increase to above 60 degrees Celsius during summer
- Cold temperatures delay the breaking of emulsions

8. ON-SITE GUIDELINES

8.1 General

Some guidelines for surface preparation and application of dilute emulsions for enrichment or rejuvenating of existing surfaces have been obtained from practitioners and are hence provided.

8.2 Road Preparation

The road surface should be clean as dust will cause the fog spray to pick up on tyres. The area to be sprayed needs to be broomed to remove the loose stone chips and dirt prior to spraying. Patching and crack sealing is usually done after the application of enrichment or rejuvenating sprays as the emulsion will act as a prime for retaining the sealer.

8.3 Dilution Process

The water used for the dilution must be potable, free of soluble salts or suspended solids ie be suitable for human consumption. It is recommended that the candidate water be subjected to a dilution test to check its compatibility with the proposed grade of emulsion. This is best done adding half a litre of the water to the required amount of emulsion and leaving it to stand in a transparent container to observe if there is any separation with time. If separation problems occur the water can be stabilised by adding hydrochloric acid for use with cationic grades and caustic soda for anionic grades. It is best to dilute the emulsion on-site immediately prior to spraying particularly if cationic spray grades are used. Stablemix emulsion is chemically the most stable grade of emulsion for diluting with water. The water must always be added to the predetermined volume of emulsion in order to avoid the formation of lumps in the diluted emulsion. This is best done by sucking the water into the bottom of the sprayer and circulating until the required dilution level is achieved. The diluted emulsion should be heated to 60 °C to facilitate ease of spraying.

8.4 Application Procedure

Once the diluted emulsion is sprayed, it must be allowed to dry with no covering, before opening to traffic. The drying time is affected by a number of factors, as described above, and adequate traffic accommodation planning is needed. It is recommended that one square metre areas be painted at various application rates to determine the application rate and to monitor the drying time.

A full trial section will:

- Indicate whether overnight closure of the road will be required in the case of inverted emulsions
- Serves as a check on the appropriate application rate

Additional guidelines are as follows:

- Any pools which develop may be blinded with washed crusher dust or coarse sand prior to opening
- Spray only when the road temperature is at least 20°C and rising.
- Do not spray if rain is expected
- Keep traffic off wet inverted emulsions and avoid using in residential areas
- Generally stop spraying by 14h00, unless the road is staying closed overnight
- In any areas of hand application, exercise care not to over-spray

- Place sand berms if steep cross-falls encountered
- No spraying should be undertaken in windy conditions

A period of 6-10 weeks should be allowed before a new bitumen stone seal is placed on a road treated with an inverted emulsion. This time could be reduced when using diluted emulsions without cutters.

9. COST-EFFECTIVENESS

The benefits of surfacing rejuvenation/ application of diluted emulsions have not yet been quantified to the extent that decisions could be based on the economic viability of these preventive measures.

The two approaches followed by some practitioners to assist in quantifying benefits could be summarised as follows:

- Area under the curve method using the standardised Visual Condition Index (VCI) as described in TRH22 (4).
  One of the problems using this method is that no benefit / improvement in the VCI can be calculated if no defects are eliminated. In the case of a diluted emulsion being added to a seal due to a too low binder content, no improvement in the VCI value is calculated.
  In addition:
  - The weight factors related to dryness of the binder and slight aggregate loss are too low to generate a sufficient increase in the area under the VCI performance curve
  - The longer term implications of rejuvenation in terms of VCI deterioration have not been quantified to define the shape of the performance curve

- Life cycle analysis using HDM4 (5)

  The opinion is held that the HDM4 models are not yet properly calibrated to analyse the true impact of surfacing seals and effects of rejuvenation in southern Africa.

  The HDM4 preventive treatments allow for two types of treatment: a fog seal and rejuvenation. The treatment is applied at the first signs of cracking or ravelling distress. The application is constrained by the user defined minimum and maximum allowable preventive treatment intervals.

  Preventive treatment is **not** applied if:

  $ACRA_b \geq 5$

  $ARV_b \geq 5$

  $NPT_a > 0$

  with

  $ACRA_b$ total area of cracking at the end of the year (% of total carriage way area)
  ARV_b area of ravelling at the end of the year (% of total carriage way area)
  NPT_a number of potholes at the start of the year

  The application of preventive treatments to the road surface delays cracking and ravelling initiation by changing the cracking and ravelling retardation factors (CRT and RRF). The procedure for the calculation of the change in the retardation factors is given. However, by definition of the two modes of application (scheduled and condition-responsive), the altering of the retardation factor should only be applied when the scheduled mode is selected.

  Thus, the cracking and ravelling retardation factors should not be altered when the condition-responsive mode is selected.

However, regardless of existing models not being able to properly quantify the benefits if rejuvenation, numerous road authorities in southern Africa consider these treatments as cost-effective to:

- Extend the life of existing seals. The general consensus is an additional three years with a diluted emulsion and even up to five years with an inverted emulsion
- Enhance the performance of reseals by pre-treatment of the old surfacing
- Prevent or reduce windscreen damage due to aggregate loss

10. TYPICAL PROBLEMS EXPERIENCED

10.1 General

The typical problems experienced by practitioners could be divided into the following:

- Poor penetration into the existing surfacing
- The emulsion breaks too slowly
- The binder stays tacky for a long time
- The emulsion develops a skin without breaking at the bottom
- Poor adhesion to aggregate
- General construction and environmental problems

Note: It is assumed for this discussion that the binder complies with the required specifications.

10.2 Poor Penetration into the Existing Surfacing

Possible causes are:

- Too dense surfacing – not sufficient voids in the surfacing for the binder
- Dirty surfacing prevents the binder from flowing into the surface voids
- Rapid breaking of emulsion thus not allowing sufficient time to flow into the voids
- Too low application rate - thus the binder is not able to 'wet' and flow into the surface voids

Note: Single seals with high texture depth are normally not too sensitive, even if the emulsion does not penetrate into the seal

Case studies:

- The road was identified for application of DE due to apparent dry and brittle binder showing oxidation/hardening cracking. Assessments were done during the dry winter months. At time of application during mid summer, no cracking was visible. Binder did not penetrate and picked up. Actions were postponed to the following winter with excellent results.
- Several situations where surfacings were dry and brittle but without sufficient voids.
- Several situations on double chip seals with sufficient voids between the larger and smaller aggregate but without access to these voids

Possible solutions

- Ensure that the surface is clean.
- Postpone action to dry season
- Prevent application during very hot temperatures if the existing binder appears to be tacky
- Lightly spray the existing surface with water to relieve surface tension
- Ensure the use of a stable grade emulsion preferably with a low viscosity
- Ensure sufficient application and dilution. Minimum of 0.8 l/m2 at a 50:50 dilution
- Ensure that sufficient voids exist to accommodate the additional binder

10.3 Emulsion Breaks Too Slowly

Possible causes:

- Recently tar-based pre-coated chips
- Very cold temperatures
- Too stable emulsion

10.4 Emulsion Breaks Too Fast

Possible causes:

- Use of rapid setting emulsion –unstable emulsion

10.5 Binder Stays Tacky for a Long Time

Possible causes:

- Too high road surface temperature > 50 °C
- Open road too early to traffic before the emulsion has broken.
- Over application resulting in high residual binder.
- Presence of flux (paraffin) in emulsion
- Fine textured surfaces are more sensitive
- Experienced often with the use of polymer modified emulsions

Possible solutions:

- Select emulsion without flux
- Select emulsion without polymer

Note: The "higher" effectiveness of a diluted polymer modified emulsion to fill surfacing cracks is still debated

- Select emulsion with higher bitumen softening point

Note: Emulsion can be manufactured with 60/70-pen bitumen

10.6 False Break

Emulsion develops a skin without breaking at the bottom.

Possible causes

- This problem has been observed with the use of polymer modified emulsions when applied in hot weather

Possible solutions:

- Select emulsion without polymer

10.7 Foaming

Foaming is a common problem when diluting emulsions. A small amount of Illuminating Paraffin (one litre) can be sprinkled on the surface of the diluted emulsion in the sprayer to reduce foaming. Also too low ratio emulsion /water

10.8 General Construction and Environmental Problems

Some of the most common problems relate to:

- Distributor and operation thereof
- On-site mixing – water quality and mix proportions (Refer Figure 6)
- Road geometry – gradients and camber causing run-off

Possible solutions:

- Check transverse distribution and leaks
- Check maximum application rates and ratio
  - Maximum 1.2 l/m2
  - Emulsion/ water ratio to 70/30 and reduce application rate to min 0.8l/m2
  - Use less stable emulsion – cationic spray70

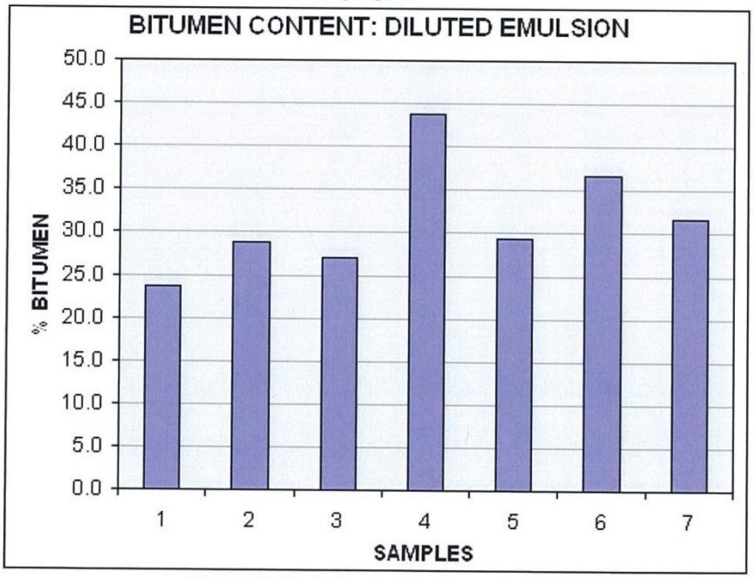

Figure 6. Variation in bitumen content.

Figure 6 shows the variation in bitumen content from samples taken on seven different projects.

Note: A 50/50 dilution of 65% emulsion was specified on all seven sites as displayed. (Target bitumen content 32.5%)

11. CONCLUSIONS

The application of diluted emulsions to extend the life of bituminous surfacings has been practiced in southern Africa for more than thirty years. Although existing performance models are considered inappropriate to quantify the benefits of these treatments, the majority of road authorities and experienced practitioners in South Africa are convinced that these measures are cost-effective and that the surfacing life could be extended by three to five years.

Different products are available on the market, each developed for a specific application. Knowledge of the purpose of development and characteristics of the different products could assist in selecting the appropriate product, mix proportion and application rate to suite different conditions.

12. ACKNOWLEDGEMENTS

The feedback and recommendations of numerous practitioners in South Africa and Namibia on this topic over the past three years is acknowledged.

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APPENDIX G.

Cementitious Mix Design

G1 Mix Design

The object of the treatment should be defined, i.e. whether it is modification to improve the properties of the material, cementation to increase the structural capacity of the pavement, or a combination of the two. This will influence the selection of the type and amount of stabiliser, the test method and also the method of construction. Figure G1 is a flow diagram of the stages in the design of stabilised pavement layers.

G1.1 Essential Elements of Mix Design

Once the object of the treatment has been defined, the essential elements of the mix design procedure are as follows:

- The most suitable type of stabiliser is selected. An investigation of more than one type of stabiliser is advantageous.
- The necessary tests are done with no fewer than three different stabiliser contents. Increments will normally vary from 0,5 to 2,0 percent and must provide sufficient information for a suitable strength/stabiliser content relationship to be determined.
- The most suitable stabiliser is selected from laboratory test results with due regard to site conditions and costs.

A mix design flow diagram is given in Figure G2

There are certain constraints that should influence the selection of the type and amount of stabiliser.

(a) For cemented materials the UCS given in Table 3 should according to TRH14[2] (paragraph 3.2.1(f) be "obtained with not more than five percent by mass of stabiliser at the specified density and at optimum moisture content" 12. Table 3 also gives maximum strengths. The object of these recommendations is to guard against the use of unnecessarily high stabiliser contents in cemented layers.

(b) Where possible, material used in cemented layers should comply with certain requirements before treatment. TRH14[2] gives the following:

Material quality before treatment	Cemented material
G2	C1
G2 or G4	C2
G5 or G6	C3 and C4

It is also stated in TRH14[2] that the grading moduli for base and subbase materials should not be less than 1,75 and 1,5 respectively. (See Subsection 3.2 of TRH14[2])

(c) Materials with properties given under (b) above are not always economically available, and when the properties of the untreated material deviate very far from the above criteria, the stabilisation process becomes more critical, durability becomes more important and greater care should be exercised in judging results.

G1.2 Borrow Pits

In all borrow pits, sufficient tests should be done to give an indication of the **variability** of the borrow pit material. One or two samples from the borrow pit should be tested by varying the type of stabiliser and also the stabiliser content. When the most suitable type of stabiliser and stabiliser content have been determined, further samples from the borrow pit should be selected and tested with the particular stabilizer and stabilizer content previously determined. The number of samples tested will depend on the size and variability of the borrow pit. The following rate of sampling is specified in the Design Manual of the Gauteng Roads Department (Gautrans)[26]:

Size of borrow pit (m^3)	Minimum number of samples
Less than 20 000	5
20 000 – 40 000	6
40 000 – 80 000	8
more than 80 000	10

(a) Materials such as basic crystalline rocks (basalt, dolerites, etc.), which are liable to weather in service, may require special consideration. Treatment to reduce the PI to SP may be desirable, and the secondary mineral content or stage of weathering should be considered in relation to the climate of the site. It is essential in all cases that the ICL be satisfied.

(b) The ICL must always be satisfied in the case of cemented materials.

(c) The coefficient of variation of the stabiliser content of random samples taken from the treated pavement layer will usually vary between 10 and 30 percent. It should not be allowed to exceed 30 percent, since this indicates poor mixing. A suitable design criterion is that not more than 10 percent of the test results, as determined from random samples, should be below the specified stabiliser content. Stabiliser content variations should be taken into consideration when the amount of stabiliser to be added in the field is specified; for example, if the coefficient of variation is 30 percent then the laboratory design stabiliser content should be increased by 1,4 to ensure that not more than 10 percent of the values from random test results in the field are below the laboratory design value. In some circumstances the limiting factor is the minimum quantity of stabiliser that can be added in practice. This applies particularly to cement-treated pavement layers where a relatively low compressive strength is required and where the material reacts well with cement.

G1.3 Initial Consumption of Lime (ICL)

The initial consumption of lime (ICL) test should be performed on all soils to be stabilised in order to determine the quantity of agent to be added. Strength tests such as the unconfined compressive strength (UCS) test have often been used to determine the stabiliser application rate but a soil with a high ICL may not be satisfied with this rate and the initial strength may be severely reduced in the field after as short a time as 14 days or even less.

Where cements or cement blends are used the ICL should be done using the actual cement to be used. This figure plus 1% is usually used in the layer, however strength tests must still be undertaken.

G1.4 Combined Stabilisers

If a plastic material must be used to provide a cemented layer, the strength specification for cemented materials may sometimes not be achieved with lime and the material may also be too plastic for satisfactory treatment with cement. The material could first be treated with lime to reduce its plasticity and then treated with cement to obtain the required compressive or tensile strength. Satisfactory results may also be obtained by the use of blends of MGBS and lime.

G1.5 Innovative Design

The recommendations summarised in the previous paragraphs are based on the criteria and the type of treatment most frequently used. A substantial improvement in the properties of almost any soil can be achieved, which is why treatment with cementitious stabilisers also lends itself to innovative design. This should always be borne in mind notwithstanding the guidelines given in the previous paragraphs, particularly in areas where good materials are costly or not available. Such treatment includes:

(a) the cement stabilisation of a large variety of sands, ranging from single-sized sands requiring high stabiliser contents to well graded sands; to provide durable base and subbase courses;

(b) the stabilisation of low-plasticity sands with lime, sometimes also with the addition of granitic soil;

(c) the treatment of plastic materials with lime to provide good pavement layers including bases;

(d) the provision of a cement-stabilised surface drain at an airport to prevent scour and erosion; and

(e) the addition of fine sandy soil to coarse gravel before stabilising.

G2 Stabilisation Cracks

Cracks, often referred to as block cracks, are a common but undesirable occurrence in many stabilised layers. These cracks are most prevalent in materials that have been stabilised with ‘early strength’ stabilisers but have been observed in lime stabilised soils as well.

These cracks are of two types:

(a) Those which occur in a relatively low clay content soil
(b) Those produced in a material with initially relatively active clay content.

In the former case, in all probability, the stabiliser was cement or a cement-slag or possibly a lime-slag mix. An excess of binder may be the cause as the relatively high early strength precludes the formation of hair cracks which give greater flexibility. The effects of temperature and moisture changes would tend to cause the layer to behave as a concrete layer might. Block cracking would tend to develop. Reduced binder content or a binder with a slower strength build-up rate may be the answer here: a lime-slag mix or a lime, if pozzolans are present, may substantially reduce the crack potential.

In the case of the active clay soil which exhibits cracking several possible solutions are suggested:

(a) Use of lime only: this procedure will give a delayed strength gain but it is essential also that sufficient time be allowed for complete modification to take place prior to final compaction.

Compaction, before modification is complete, even with lime, may produce a locked-in shrinkage potential which will result in cracks being formed as the clay dries out.

(b) Add lime only at first: add slag or cement later. This procedure will ensure that modification is complete before any serious strength build-up occurs. The later addition of cement or slag can then be followed by rapid compaction without fear of a locked-in shrinkage potential. Cracking by soils treated in this manner has been very successfully reduced, if not eliminated entirely, in many road works in the Republic. The additional cost of double mixing should, however, be taken into account.

(c) If the moisture content of the soil being compacted is kept below the shrinkage limit (SL) for the material the cracking potential should be virtually eliminated, the argument being that insufficient moisture is present (as it is below the SL) to cause any shrinkage cracking. The practice has also shown positive results in many works where active clay soils are involved. It must be stressed however that the reduced moisture content will most likely be below the optimum for the economical compactive effort necessary to produce the specified density and a greater effort will be required. The extra cost for this additional effort should be taken into account. As a general guide, the moisture content at which compaction should be undertaken is when the degree of saturation is not more than 70 percent.

G3 Problem Soils

The following soils have been known to give stabilisation problems in South Africa and are listed in order to encourage an awareness of the fact that although most soils lend themselves to stabilisation there are a few “dangers” which should be recognised:

(a) Lateritic soils or ferricretes: these soils contain a high iron concentration and the ferric ions may not readily be replaced by calcium. Consequently these soils are best stabilised with cement or cement-slag mixes. Even then, it has been found that the ferricrete nodules are sometimes relatively soft and stabilisation only cements the nodules on the outside with the result that, when an the ITS of the cemented material is determined, it is impossible to meet the design requirements.

(b) Kalahari or uniformly graded dune sands: these soils are generally very low in pozzolans and are consequently poor soils for treatment with lime. Because of their relatively poor grading, stabilising with cement or cement slag mixes is necessary and even then relatively large application rates may be necessary due to the very high surface area to be cemented.

(c) Calcretes: It is often an erroneous assumption that calcretes which are limestone cemented materials need not be stabilised. These deposits are calcium carbonate and not lime (calcium hydroxide). They often have relatively high plasticity and need to be stabilised in many instances; because of their high calcium content cement and slag is often better than lime as the stabiliser.

(d) Structurally weak particle soils: although these soils may be satisfactorily stabilised they should always be tested for durability by means of the wet-dry brushing test and / or ITT to ensure that not only strength requirements but also their resistance to erosion is satisfied.

(e) Micaceous soils (such as some granites): these soils have a springy or spongy action and compaction is difficult. They have a high water demand and cement or cement slag should be used as stabiliser. As a precaution, should mica be readily visible, the soil should preferably not be stabilised.

(f) Soils containing sulphides: soil containing minerals such as pyrite (golden), marcasite (silver), chalcopyrite or copperpyrite (iridescent) should be avoided if these minerals are in excess of about 1% by volume. These sulphides can form sulphuric acid and sulphate salts which destroy the stabiliser and its cementing effect.

(g) Soils containing soluble salts: soluble salts, generally sulphates (SO_4) are often present in soils in the dry areas of Southern Africa where Weinert's N is greater than 5, (Weinert, 1986).

These salts tend to form acids and produce undesirable crystallisation. Although imported soils in fills may tend to lose their salts, in-situ soils or low-lying fills may become recharged due to water ingress. Soils of this nature should not be stabilised if the sulphate content is greater than:

(i) 0,25 percent if the PI or P0,002 is greater than 12%
(ii) 1,0 percent if the PI and P0,002 are each greater than 12%

(h) Soils with organic impurities: soils containing materials such as lignites, coal, humus, sugar or sugar waste should preferably be stabilised with lime only and in any event avoided if these impurities are in excess of 1% by volume.

(i) Very highly plastic soils: soils which exhibit a high plastic limit (in excess of 45) should not be stabilised with cementing materials such as portland cement or slag. Modification only with lime should be considered but even then large application rates, in excess of 6 to 8 percent by mass, may be required.

(j) Highly acid soils: soils with a low pH value resulting, say, from acidic rocks or with a high humus content exhibit a high initial lime demand. These soils will require large application rates. Soils showing an ICL in excess of 3 percent should generally be avoided.

(k) Soils contaminated with industrial wastes: these soils may contain chemicals, which may react adversely with any stabiliser. Particularly aggressive are wastes containing acids, organic impurities, sulphates and even bituminous by-products and oils. Tests by means of chemical analysis and the wet-dry brushing should be performed on all soils near industries likely to cause any contaminations. It may often be profitable to avoid any attempt at stabilising such suspect soils and to import from a more stable source, particularly as these contaminated soils are likely to be confined to a small area.

(l) Soils exhibiting high variability: when field tests show erratic results, soils of a variable nature are indicated. Such soils may prove to be problematic particularly in densification and any form of stabilisation, which is not a cheap operation. They should be avoided wherever possible.

G4 Carbonation

An important reaction that does not fall within the category of modification or cementation involves the absorption by lime and cement of carbon dioxide (CO_2) from the air to produce calcium carbonate. Not only does the formation of carbonate rob the stabiliser of its active component but it also deters pozzolanic action and so reduces normal strength gains. This reaction occurs also with cement/slag and lime/slag stabilisation.

Because of this possible reaction care should be taken to prevent the stabiliser from being carbonated by carbon dioxide from the air. After mixing the lime layer should be sealed as soon as possible. It should be borne in mind, that ground water may contain high carbon dioxide concentrations.

It is also important to note that destructive carbonation is at a minimum when the pH is high. The records show that in most cases where destructive carbonation has taken place the pH was generally below 12 although it could also be that the ICL has not been satisfied with enough reserve $Ca(OH)_2$ to maintain a high pH for the stabilisation process to be completed.

There is strong evidence that carbonation is not always destructive. Tests on stabilised layers, more than ten years old, have shown severe carbonation, yet no loss in strength was evident.

G5 Curing

When soil is stabilised in such a manner that lime modification is allowed to take place before final compaction, this period of time may be termed an "initial curing period".

Before initial curing and during initial mixing the moisture content should be increased to above optimum for the following reasons:

(a) The modification reaction is ionic, which means the lime must be in solution

(b) Lime is very insoluble in water and becomes more insoluble with rising temperatures

(c) In South African climatic conditions are generally ideal for rapid evaporation losses which would stop the reaction if the layer became dry.

During initial curing the layer should be lightly compacted and kept continuously moist to reduce moisture loss and the possibility of carbonation of the lime or cement.

After final compaction curing continues provided that the layer is kept moist and free from the ingress of carbon dioxide. The completed stabilised layer may be covered immediately with a further layer which would ensure a continued high moisture environment or it may be sprayed with a prime coat or kept continuously wet by means of fog spraying until finally covered. Alternate wetting and drying should be avoided.

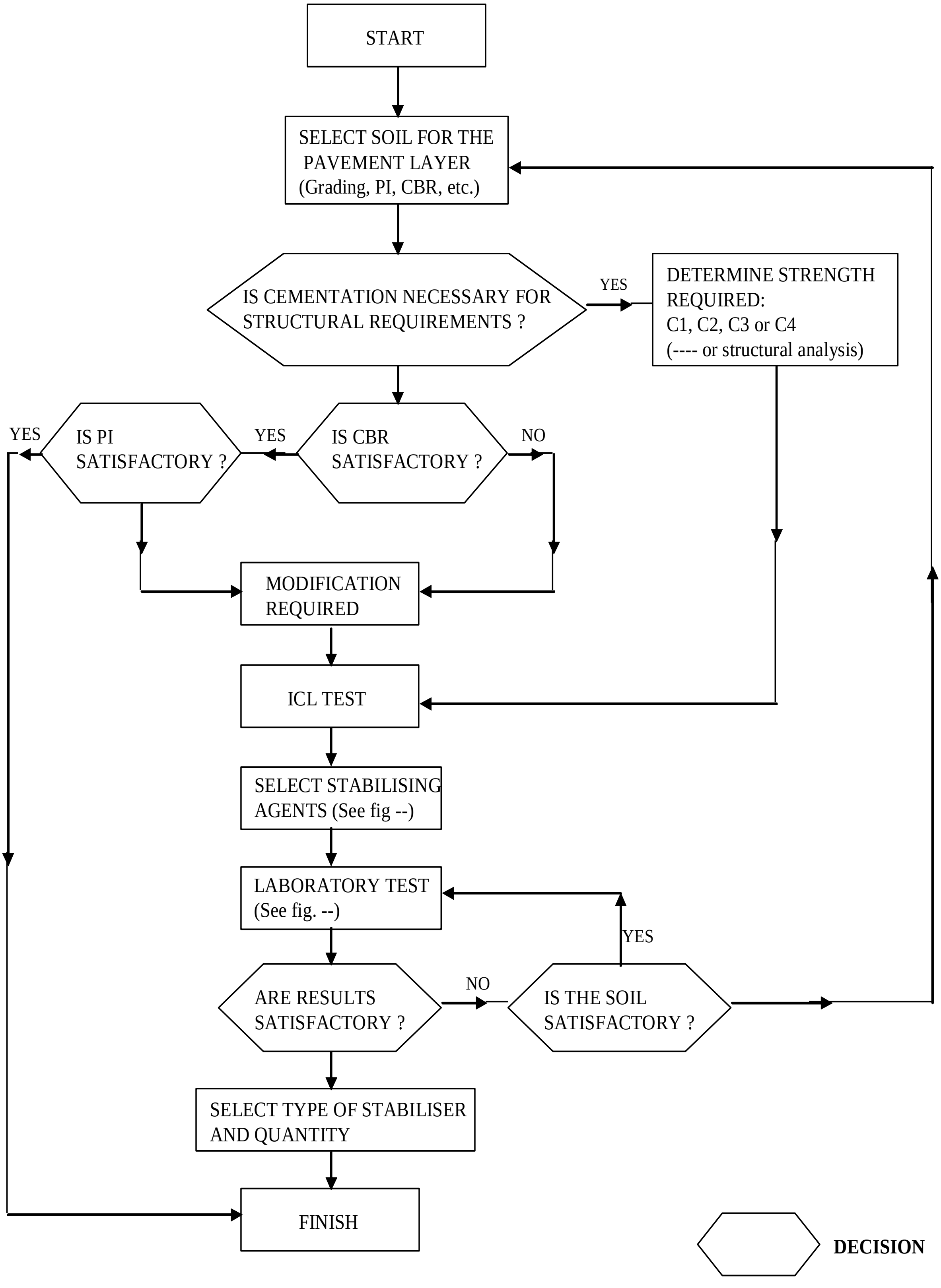

FIGURE G1

Stages in the design of stabilised pavement layers

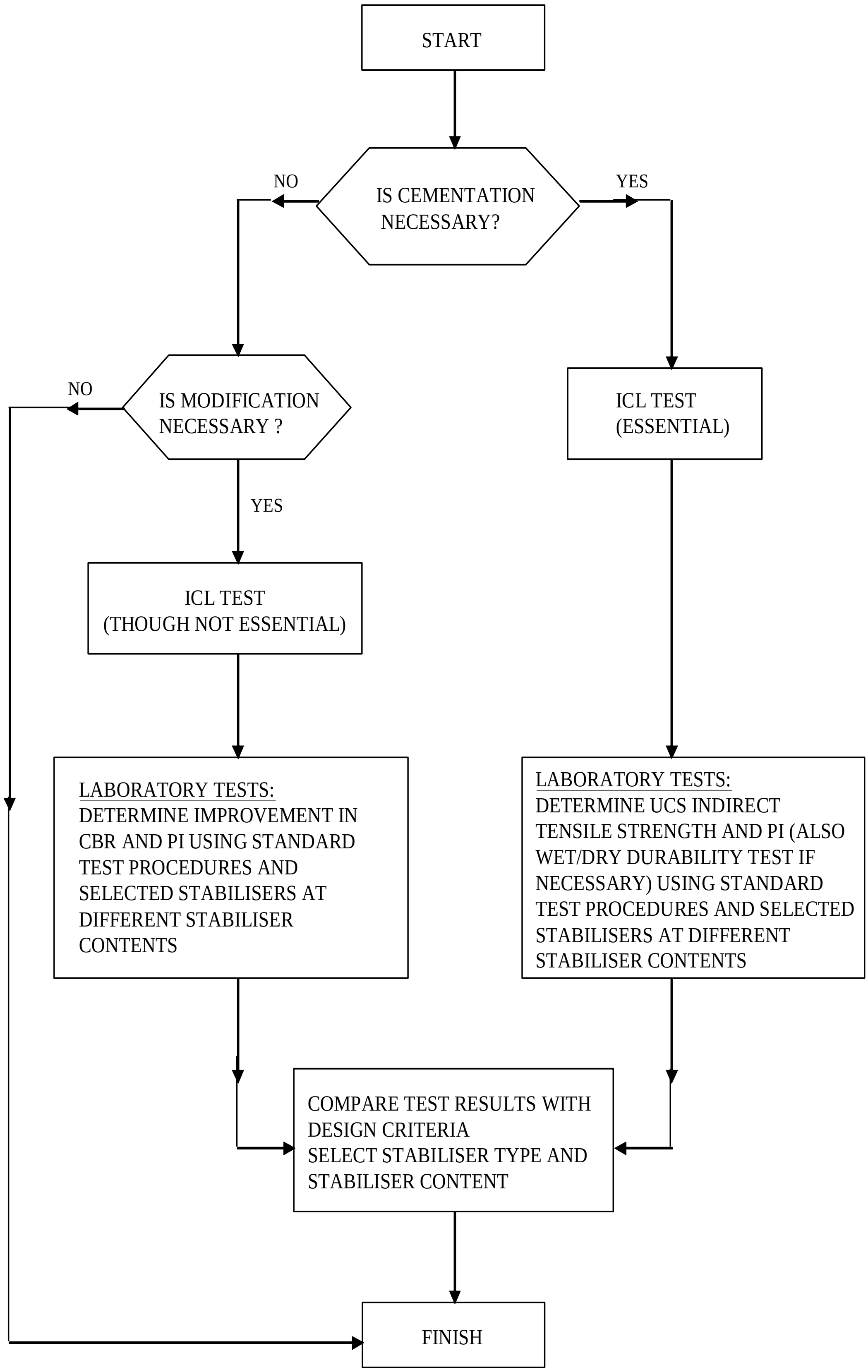

FIGURE G.2

Essential elements of mix design