A Knowledge Based Structural Design Method for Pavements Incorporating Bituminous Stabilized Materials

Technical Memorandum

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1 BACKGROUND AND INTRODUCTION

Emulsified bitumen treated materials have been successfully used in SA for many years. Two guidelines for their use are available: Manual 14 (SABITA, 1993) and Manual 21 (SABITA, 1999). Foamed bitumen is a newer technology than emulsified bitumen, and has not been as widely used in South Africa. In 2002, the Interim Technical Guideline: “The Design and Use of Foamed Bitumen Treated Materials” (TG2) (Asphalt Academy, 2002) was released by the Asphalt Academy. The need for this document was identified by the broader industry in light of the increased use of foamed bitumen, particularly for in-situ recycling, in South Africa.

In the interim TG2 publication, it was acknowledged that the guidelines were representative of current best practise, and would need updating. Limitations in the document were identified, and since 2002, some aspects of the document - particularly the mix and structural design sections - have been highlighted. It was also felt that the document does not clearly illustrate the benefits of foamed bitumen treatment. For the structural design section in TG2 an area of concern with the structural design models was the lack of validation using field performance data, and apparent differences between long term field behaviour, behaviour under accelerated testing and predicted performance from laboratory testing. In addition, it was felt that limitations in the use of mechanistic-empirical design methods can lead to the inappropriate design of foamed bitumen pavements.

In parallel with the work on foamed bitumen and TG2, at the time of the release of TG2 it was recognised that an equivalent guideline on emulsified bitumen treated materials was required to update Manuals 14 and 21 and to place foamed and emulsified bitumen on an equal footing. A technical guideline document for emulsified bitumen treated materials was therefore drafted and subjected to a review by industry representatives, but was never published. This guideline has been referred to as TG(X). TG(X) is very similar to TG2, and therefore the areas that need updating in TG2 also need updating in the TG(X) guideline.

Because of the many similarities between foamed and emulsified bitumen treated materials it was decided that it would be appropriate to publish one guideline document for Bitumen Stabilized Materials (BSM) and that this guideline would cover both material types. Thus, a project was initiated to develop such a guideline document. The project was designed to address two key deficiencies in existing guidelines: (a) structural pavement design; and (b) mix design.

In view of the considerable scope of the project, it was planned in several phases, starting with an Inception Study (Phase 1) completed during 2006. The second phase of the project is currently in progress and is scheduled for completion towards the end of 2007. This technical memorandum forms part of the second phase of the structural pavement design component of the study. The work proposal for the structural design component is included in Appendix A.

1.1 PROJECT OBJECTIVES

The objectives of this project, as outlined in the project proposal, are as follows:

- To expand the Long Term Pavement Performance (LTPP) database compiled during the Inception Study to a sufficient level to enable the development of classification based design guidelines.
- Develop and populate a classification based design method.
- Peer review the recommended design guidelines.


1.2 **SCOPE AND CONTEXT OF THIS DOCUMENT**

This memorandum describes a new design method for pavements that incorporate bitumen stabilized materials. As such, this document addresses elements of the structural design component of the study, and in particular the second objective noted above, namely the development of a classification based design method for pavements that incorporate bitumen stabilized layers.

It is important to note that this document forms part of an ongoing study. As such, several other components of the study are still in progress and are likely to impact on the eventual implementation of the method that is outlined in this memorandum. The following three components of the study are of special relevance:

- **Materials Classification System Development**: this study forms part of the structural design component of the overall project. The materials classification system was developed to provide designers with a consistent methodology for determining the inputs required by the pavement design method outlined in this memorandum. At present, the materials classification system is ready for pilot testing, which is planned for the second part of 2007. The basic principles and current status of the materials classification system is outlined in Jooste et al. (2007).

- **Summary of LTPP Database**: the collection of field performance data for pavements that involve bitumen stabilized layers formed a key element of the study. Significant efforts were expended to compile a comprehensive database of LTPP sections that involve bitumen stabilized materials. This database forms the basis for the structural design method described in this memorandum. Details of the sections included in the LTPP database can be found in Long and Jooste (2007).

- **Mix Design Component**: this vital element of bitumen stabilization is addressed in a separate study, and is thus not covered by this memorandum. The findings of the mix design component of the study are expected to be published during 2008.

It is vital for readers to note that the design method proposed in this document, while believed to be sound in concept, still needs to be pilot tested. This testing is scheduled to take place during 2007, after which the refined method will be published as part of the finalized guidelines.

1.3 **REPORT STRUCTURE**

- Section 2 of this report outlines and motivates the basic approach adopted in the development of the proposed pavement design method.

- Section 3 provides a discussion of the fundamentals that were used as points of departure in the development of the pavement structural design method. Key concepts are introduced and explained in detail.

- In Section 4, a detailed methodology is provided for the calculation of the Pavement Number (PN), which is a key element of the proposed design method. In essence, this section describes the manner in which the fundamentals outlined in Section 3 are implemented in the proposed design method.

- Section 5 provides a discussion of the observed long term behaviour of bitumen stabilized layers. The discussion is based on observations from the LTPP database, and also postulates a behavioural model for bitumen stabilized materials, as commonly constructed in southern Africa.

- Section 6 describes the calibration of the relationship between Pavement Number and the observed structural capacities of pavements in the knowledge base.

- Section 7 discusses the development of design criteria for the Pavement Number. Details of the development approach are discussed, and a validation of the criteria is provided through reference to structures in the LTPP database, and of structures tested with the Heavy Vehicle Simulator.
Section 8 addresses peripheral elements that should form part of the final design guidelines. These elements include practical aspects such as the selection of an appropriate surfacing as well as problem materials and construction processes. The section highlights the need for a risk assessment system which will alert designers to risk factors that enter in the materials design, pavement design and construction processes. A conceptual example of such a risk assessment system is provided, and recommendations are made for the refinement of the system to facilitate inclusion in the final design guidelines.

Section 9 provides a summary with recommendations for refinement and implementation of the findings.

It should be noted that this report is classified as a Technical Memorandum, as defined by the documentation guidelines of the Gauteng Department of Public Transport, Roads and Works. As such, the main purpose of the document is to record technical processes and data, and not to present a formal, finished document for general distribution.
2 A PAVEMENT INDEX APPROACH

The design method developed during this study relies on an index to quantify the long term load spreading capacity of a pavement. The index is called a Pavement Number (PN) and is used to determine whether a pavement structure is appropriate for a given traffic intensity and confidence level. The PN is similar to the Structural Number (SN) widely used in AASHTO design methods published before 2000 (AASHTO, 1986). As will be shown in later chapters, the PN is calculated using the layer thicknesses and assigned material classes. An empirical relationship between the PN and observed performance of more than 80 pavement structures provides the basis for using the PN to assess design capacity.

There are five reasons why an index-based approach was used in this study, as opposed to the more popular and sophisticated Mechanistic-Empirical (ME) design method. These reasons are:

(i) Ease and Transparency of Calibration and Validation
As will be shown later, the pavement performance data gathered as part of this study provides details of the overall performance of each of the pavements that were studied. These data contain details of the construction, traffic and performance of each pavement section. As such, the available data provides an adequate description of the performance history of the pavement as a system. However, the available information lacks the necessary details to determine the key element of a traditional ME methodology, namely the deterioration history of individual layers.

This lack of detailed information on layer deterioration is further complicated by the fact that relatively few of the pavements studied showed structural failure. Thus, models describing deterioration of individual layers to failure cannot be derived from the available information, except through subjective interpretation and considerable extrapolation. By comparison, the index-based approach – which provides a single indicator to summarize the structural capacity of the pavement system – lends itself well to calibration using the performance information of a pavement system (as opposed to individual layers).

Thus, compared to the traditional ME method, the index-based approach is easier to calibrate and validate using either accelerated or long-term pavement performance data. Furthermore, the coarseness of available performance data necessitated a more practical approach to link the pavement capacity to observed performance. Many variants of the ME design method have never been validated against actual long term pavement performance (LTPP) data, and thus remain largely theories of pavement performance*. By contrast, calibration and validation forms an integral part of the development process of the index-based approach as proposed here.

(ii) Use of Well-Defined, Unambiguous Inputs
As will be shown in later chapters, the PN is determined using material classes which can be confidently obtained as part of a routine rehabilitation investigation. By comparison, the ME method requires the use of the ill-defined resilient modulus. This parameter cannot be uniquely determined for all stress states, and values obtained vary greatly depending on the experience of the designer and on the method of measurement, at least three of which are being used by practitioners in South Africa*. In 2005, one of the authors asked a prominent South African pavement researcher how many studies had been dedicated to the systematic verification of the South African ME method using long term pavement performance data. His answer was simply “none”. The method had then been in use for more than 15 years.

* Practitioners obtain elastic moduli from published tables, backcalculation of Falling Weight Deflectometer (FWD) deflections and backcalculation of Deflectograph deflections. Some elements of the SA Mechanistic Design Method were developed using moduli determined from laboratory tests and from backcalculated deflection values measured with an HVS.
(iii) Robustness
The index-based approach is more robust than the ME method and cannot be easily manipulated. As such, the method is not only suited to specialists, but is also accessible to practitioners with different levels of design experience.

(iv) Better Coupling Between Design and Contract Specifications
The inputs of the index-based approach proposed here consist of, or are dependent on, parameters which are the same as those that are used in contract specifications (e.g. grading, plasticity index, CBR, etc). By contrast, the ME method assessment, in its present form, is highly dependent on the assumed elastic modulus, a parameter which seldom forms part of contract specifications.

(v) Transparency and Educational Value
Unlike the ME method, which cannot be implemented without using specialized computer programs, the index-based approach is a transparent, straightforward approach that can be implemented by means of a spreadsheet or even a paper worksheet. Also, it will be shown in later chapters that the method of calculation explicitly incorporates basic rules of pavement behaviour, and thus serves as an educational tool in its own right.

In the following sections, we provide brief examples of how simple pavement indices can be correlated with pavement capacity. We then explain conceptually how a pavement index can be used to develop a design method using field observations. Finally, we look at some of the disadvantages of the index-based approach, and how these can be addressed in a knowledge-based design method.

2.1 EXAMPLES OF PAVEMENT INDICES

2.1.1 The AASHTO Structural Number
The AASHO road test of the late 1950’s and early 1960’s led to the well known and widely used Structural Number (SN) which is used to quantify the protective and load spreading effects of all pavement layers combined. SN is calculated by the following equation (AASHTO, 1986; Huang, 1993):

\[ SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \]  

(Equation 2.1)

Where: 
- \( a_{1,2,3} \) = the layer coefficients for layers 1, 2 and 3 respectively;  
- \( D_{1,2,3} \) = the layer thicknesses (in inches) of layers 1, 2 and 3, respectively, and  
- \( m_{2,3} \) = the drainage coefficients for layers 2 and 3 (non-surfacing layers).

Equations were developed (Van Til et al., 1972) to relate the layer coefficients for different classes of materials (such as asphalt surfacings, unbound base layers, etc.) to the layer stiffness. Similarly, the drainage coefficients can be obtained for different combinations of rainfall intensity and drainage quality.

As simple as it is to calculate, and as outdated as it seems, SN is well-correlated with the expected performance of South African pavements. To illustrate this, the structural number was calculated for the granular base pavements with cemented subbases for Category A roads in the TRH4 catalogue (TRH4, 1996). An example calculation for one of the structures is shown in Table 1.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>TRH4 Material</th>
<th>AASHTO Material Assumed</th>
<th>Assumed Stiffness (MPa)</th>
<th>AASHTO Layer Coefficient</th>
<th>Drainage Coefficient</th>
<th>SN Contribution</th>
<th>Structural Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>AC</td>
<td>Dense Graded Asphalt</td>
<td>3000</td>
<td>0.45</td>
<td>1.00</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>G1</td>
<td>Untreated or Stabilized Base</td>
<td>650</td>
<td>0.26</td>
<td>0.80</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>C3</td>
<td>Untreated or Stabilized Base</td>
<td>2000</td>
<td>0.38</td>
<td>0.80</td>
<td>2.39</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>G7</td>
<td>Granular Subbase</td>
<td>120</td>
<td>0.12</td>
<td>0.80</td>
<td>0.59</td>
<td>4.74</td>
</tr>
</tbody>
</table>

Table 1: Example Calculation of a SN
Figure 1 shows the calculated SN values plotted against the maximum allowed structural capacity for each structure. This figure shows a strong correlation between SN and capacity, despite the simple formulation of the SN. As expected, the SN versus capacity plot shows a strong exponential trend, which is also reflected in the TRH4 catalogue structures. For example, for dry regions, the only difference between the structure recommended for 10 to 30 million standard axles (msa) and the one recommended for 30 to 100 msa is 50 mm of cemented subbase (C3 material*).

It is also of interest to note that, by incorporating a well-selected drainage factor for pavements in the wet regions, the SN of the pavements in the wet and dry regions could be matched well in two out of three points (the structures for wet and dry regions for 10 msa could not be well matched).

![Figure 1: SN versus Allowed Structural Capacity for some Granular Base Structures in TRH4](image)

2.1.2 A Pavement Number (PN) Based On Effective Layer Stiffness

The obvious correlation between the AASHTO SN and allowed pavement capacity illustrates the potential for using a pavement index to evaluate the appropriateness of a design for a given traffic demand. It is important to note that the SN is just one type of indicator. Other pavement indices can easily be developed and calibrated for southern African conditions. Analysis of SN versus allowable traffic quickly shows that the selection of an appropriate stiffness for different material types is a key element in determining an effective pavement index.

Using this finding, a similar index can be determined using only layer thickness and layer stiffness as parameters. The index can be made more "intelligent" by determining the stiffness of each layer based on the material class, coupled with the ratio (called the Modular Ratio) between the layer stiffness and the stiffness of the supporting layer. Apart from addressing stress sensitivity in unbound materials, in which the material stiffness is highly dependent on the stiffness of the support, such a scheme also simulates fatigue in cohesive layers. It does so by coupling the long term stiffness of cohesive materials, such as asphalt or cement treated materials, to the stiffness of the support. An example of such a calculation is shown in Table 2.

---

* All material codes (e.g. G1, G3, C3, AC) used in this document refer to the South African material classification system for road-building materials, as defined in the TRH14 guideline (TRH14, 1985).
For the example shown in Table 2, the PN contribution of each layer is calculated by multiplying the layer thickness with the layer stiffness and drainage coefficient, and dividing by a scaling factor (10,000 in this case). The PN is then calculated by adding the contributions of each layer.

Table 2: Example Calculation of a PN Based on Layer Stiffness and Thickness

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>TRH4 Material</th>
<th>Stiffness Determination Rules</th>
<th>Assumed Long Term Stiffness (MPa)</th>
<th>Drainage Coefficient</th>
<th>PN Contribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>AC</td>
<td>5 times support stiffness, to 3000 MPa maximum</td>
<td>3000</td>
<td>0.80</td>
<td>7.2</td>
</tr>
<tr>
<td>150</td>
<td>G1</td>
<td>2 times support stiffness, to 650 MPa maximum</td>
<td>650</td>
<td>0.80</td>
<td>7.8</td>
</tr>
<tr>
<td>200</td>
<td>C3</td>
<td>10 times support stiffness, to 2000 MPa maximum</td>
<td>1200</td>
<td>0.80</td>
<td>19.2</td>
</tr>
<tr>
<td>150</td>
<td>G7</td>
<td>Selected Layer (assumed fixed stiffness)</td>
<td>120</td>
<td>0.80</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Figure 2 shows the calculated PN values, using the calculation scheme shown in Table 2, for Category A road granular base pavements with cemented subbases in the TRH4 catalogue. The PN values are again plotted against the maximum allowable traffic for the pavement in question. Figure 2 again shows a clear correlation between the simple PN index and the allowable traffic. As before, a close agreement between the PN values for pavements in wet and dry regions could be obtained through the selection of a suitable moisture factor for pavements in the wet regions.
2.2 **LINKING A PAVEMENT INDEX TO OBSERVED PERFORMANCE**

In the preceding section it was showed that even simple pavement indicators are reasonably correlated to expected or allowed pavement performance. In this section, we discuss how a relationship between a pavement index and observed pavement performance can be developed using field observations and design catalogue data.

As will be shown in later sections, it is possible to calculate a pavement index, or number, by using basic parameters such as the thicknesses and material classes of the pavement layers. The constants or rules that govern the index calculation can be calibrated using pavement structures for which the performance limits are known with reasonable certainty. Pavement structures in the TRH4 catalogue can typically be used for these purposes.

Since design catalogue structures are generally quite conservative, the relationship between the pavement index and the allowed traffic loading provides the outer limits of a *confidence frontier*. The relationship can be improved and refined by adding pavement structures for which the layer configuration, performance and estimated traffic accommodated are known. Such information can be obtained by studying available as-built information, data in rehabilitation reports and those in pavement management systems.

The link between the observed pavement performance (for studies of actual pavements) and allowed pavement capacity (for catalogue structures), can be visualized by plotting and analyzing the available data as shown in Figure 3. This figure is similar to that shown in the preceding examples, but now includes data from actual field observations. These field observations can be either LTPP data or Heavy Vehicle Simulator (HVS) data.

Naturally, not all of the pavements obtained from field studies will have failed, and some may have been in service for only a few years. These pavements are represented by the green dots in Figure 3, which are shown with upward arrows to indicate that the traffic accommodated is still increasing. For example, point A, shown on the lower right hand side of Figure 3, represents a relatively new pavement with a fairly high index value. We do not know when this pavement will fail. However, looking upwards from point A we can see there are three other pavements in the data set with a similar pavement index (one failed, one in warning and another in good condition). These three observations allow us to state with some confidence what a minimum expected performance level (i.e. cumulative traffic before failure) would be for the pavement represented by point A.

It is important to note that the lines shown in Figure 3 are not trend lines (i.e. lines fitted to the data) and they are also not transfer functions (i.e. lines indicating when failure would occur). Rather, these lines are imposed on the data using domain knowledge and assessment of the general trend exhibited. As such, the lines represent the frontiers of confidence or experience for successful implementation of pavements with different pavement index values. From this perspective, it becomes easy to see how a transparent, straightforward design method could be developed with information such as that shown in Figure 3.

Rather than trying to fit trend lines to points of failure (as in the development of the ME method), the data set shown in Figure 3 can be used to establish frontiers of confidence for the performance of pavements with different ranges of index values. Figure 4 illustrates this concept.
A Pavement Index Approach

Cumulative Traffic Accommodated or Allowed

Pavement Index Value

Figure 3: Concept for Establishing a Relationship between a Pavement Index and Field Performance

Figure 4: Interpretation of Observed Pavement Performance within an Index Range
As shown in Figure 4, within the index range X to Y, there is little or no trend between the index values and the observed pavement performance. This is to be expected, given the coarseness of the available information and the highly variable (perhaps even chaotic) performance of pavements with similar configurations. Yet the data are still useful. For the index range X to Y we have the observed performances of seven pavements, and one catalogue structure. The observed or allowed performance of these eight structures allow us to state upper and lower frontiers of observed or expected performance, represented by Na and Nb in Figure 4.

Thus we can state that, if a pavement has an index value in the range X to Y, it is highly unlikely that the pavement will fail before the number of cumulative standard axles exceeds Na. It is also unlikely that failure would occur before Nb is exceeded, as only one out of seven structures failed before this limit, and two are still performing well beyond this limit.

It should be noted that, for the data set shown here, a similar interpretation cannot be obtained simply by a brutal statistical approach in which trend lines with different confidence levels are forced through the data. This is because the data points shown in Figure 4 do not all represent the same situation. Some points denote failure, most do not.

The preceding paragraphs presented an approach for developing a pavement design method using field data and a pavement index. In Sections 3 to 7 we will implement this approach using data collected as part of this study, coupled with structures from design catalogues.

2.3 DISADVANTAGES OF THE INDEX-BASED APPROACH

“Seek simplicity, but distrust it” (Alfred North Whitehead)

The advantages of the index-based approach were discussed in the introduction to this chapter. Naturally, no method is perfect, especially not a simplified approach which attempts to capture all the complexities of pavement behaviour in a single number. More specifically, in its simplest form (e.g. the AASHTO SN approach), the index-based approach suffers from three important disadvantages:

(i) Non-Uniqueness of the Calculated Index

Perhaps the most serious disadvantage of the index-based approach is that different pavement configurations can have the same pavement index. Since the pavements have the same index, the assessment of performance will be the same. For example, if the SN is calculated for a pavement with a crushed stone (e.g. G2) base over a cement stabilized (e.g. C3) subbase, then the same SN value will be determined if the base and subbase layers are changed around. Yet we know that the performance of a G2 over C3 pavement will be different from that of a C3 over G2 pavement.

*This disadvantage can be addressed* by making the index more intelligent. This can be done by incorporating basic rules of pavement design, as well as region and material specific experience, in the calculation of the index. For example, in the case noted above, the use of a modular ratio to determine the effective stiffness of the material will mean that the stiffness of a G1 base over a C3 subbase will not be the same as that of a G1 base placed over a weaker support (as will be the case if the G1 base is placed below the C3 layer). Thus if a rule that forces modular ratios to be observed

* Sophisticated statistical techniques to analyze and model complex failure-time data that incorporate missing or censored data exist. However, the approach recommended here is a more straightforward engineering assessment of the available information, as in this example.

* All material codes (e.g. G1, G3, C3, AC) used in this document refer to the South African material classification system for road-building materials as defined in the TRH14 guideline (TRH14, 1985). A G2 material is a good quality crushed stone and a C3 material is a cement-stabilized material with an Unconfined Compressive Strength (UCS) greater than 1500 kPa.

* The modular ratio, as used in this document, denotes the ratio of the stiffness of one layer, relative to the stiffness of the supporting layer. Thus if the stiffness of a base layer is 300 MPa, and the stiffness of the supporting subbase is 150 MPa, then the modular ratio of the base layer will be 2.0. This aspect will be discussed in detail in Chapter 3.
is incorporated into the calculation of the pavement index, then the two cases noted above will no longer have the same pavement index.

It is thus possible to eliminate this most important of pitfalls of the index-based method simply by incorporating fundamentals and experience. In the next chapter, we will show in detail how this can be effectively achieved. Furthermore, by explicitly incorporating fundamentals and experience into the design method, the inexperienced designer is almost forced to learn the fundamentals of pavement behaviour as part of the design process.

(ii) Insensitivity to Placement of Weak Layers
Another disadvantage of the index based approach is its apparent insensitivity to the position of weak layers in the pavement system. For example, it is possible to obtain a relatively high structural number by placing a thick but soft material at the top of the pavement. Such a pavement will clearly not be able to withstand high intensity loading, despite the fact that the SN value is high.

As with item (i) above, this disadvantage can be addressed simply by incorporating a few sound design rules that limit the quality of base material and surfacing type for specific traffic classes. This can be done by including a factor that reflects the confidence (based on experience and field evidence) that certain base types are able to accommodate a specific design traffic.

(iii) Limited Analytical Capability
There is no doubt that the index-based approach does not have the same analytical power of the ME method. In the hands of an experienced practitioner, the ME method offers far greater potential to determine why a pavement structure will fail than the index-based approach. In the index-based approach, a limited analytical capability can be achieved by analyzing the contributions of different layers toward the overall pavement index. Such an analysis can provide an indication of pavement balance (similar to the interpretation of a Dynamic Cone Penetrometer curve) and where failure is most likely to occur. However, this limited analytical capability cannot compare to that of the ME method.

It is for this reason that the main objective of the index-based approach, as presented here, is to obtain a reliable indication of expected or allowed structural capacity for a proposed pavement design. For high profile design situations, the index-based method can be combined with the ME approach to provide improved analytical capability.

A fourth, but less important, disadvantage of the index-based approach is the negative perceptions held toward the approach by researchers and some practitioners. An index-based approach does not have the same attraction of sophistication and modernity that the ME method has. The complexity of the ME approach suggests a thoroughness which belies the many questionable assumptions, extrapolations and simplification of data that lie hidden in the darker recesses of the methodology.

It should also be noted that proposed use of the index-based approach is not unique in this era. The TRL guidelines for the use and specification of cold-recycled materials, published as recently as 2004, makes use of a pavement index, called a Structural Equivalence Number (SEN) to determine suitable designs for different road categories (Merrill et al., 2004).
3 THE BASIS FOR AN INTELLIGENT PAVEMENT NUMBER

In the previous section, aspects related to the use of an index-based approach to structural capacity assessment were discussed. It was shown that even simplified pavement indices such as the AASHTO Structural Number are generally well correlated to expected structural capacity. The disadvantages of the index-based approach were discussed, and it was noted that two of the main weaknesses of the approach (i.e. non-uniqueness of the index and insensitivity to placement of weak layers) can be overcome through the incorporation of basic principles of pavement behaviour and performance.

In this section, the challenge of developing an intelligent quantifier of pavement structural capacity is discussed in more detail. The main objective of this section is to provide a basis for the development of a heuristic, or knowledge based, Pavement Number (PN). As such, the discussion will focus mostly on general concepts. Particulars of the calculation of the PN will be presented in Section 4.

3.1 A KNOWLEDGE BASED APPROACH

The development of the PN-based design method, as described in this document, relies on a heuristic*, or knowledge based approach. In this approach, a formulation for a Pavement Number was developed using 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 first step in the development of the PN-based approach was thus the identification of general rules-of-thumb which determine the manner in which pavement systems, and individual pavement layers, will behave and perform. These rules were then quantified by means of constants or mathematical functions designed to mimic the identified rules-of-thumb in a general way. The quantified rules were then combined in a methodology used for calculating a PN value.

Using the adopted methodology, the PN value was calculated for several pavements extracted from the TRH4 catalogue (TRH4, 1996), and for which the structural capacities were known with some certainty. The calculated PN values were then correlated with the structural capacities and the underlying rules and functions were systematically adjusted to optimize the correlation between PN and structural capacity (this first order calibration process is described in more detail in Section 6).

In the development of a methodology to calculate the PN, many different approaches were considered. Some of these were fairly complex, and involved repeated calculations with a linear elastic program. Other approaches were much simpler, and proceeded along the lines of the AASHTO Structural Number. The approach that was eventually adopted was the one that showed the best correlation between the calculated PN and the observed structural capacity of pavements in the available data sets.

The following subsections present a discussion of the basic rules-of-thumb underlying the proposed method for calculating PN. These rules-of-thumb reflect well-established principles of pavement behaviour and performance, and thus no attempt is made to rigorously reference or prove the adopted rules-of-thumb. Instead, the discussion aims at clarification of terminology and patterns, to enable the reader to understand how each rule is integrated into the calculation of PN. The proof of the validity of the rules is implicit in the success or failure of the resulting model, which is validated in Section 7.3.

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* Hopgood (2001) used the following quote from Barr and Feigenbaum (1986) to provide a general description of the term “heuristic” as it applies to numerical search methods: “A heuristic is a rule of thumb, strategy, trick, simplification, or any other kind of device which drastically limits search for solutions in large search spaces. Heuristics do not guarantee optimal solutions; in fact they do not guarantee any solution at all; all that can be said for a useful heuristic is that it offers solutions which are good enough most of the time”.
3.2 ADOPTED RULES-OF-THUMB

The following rules-of-thumb were adopted as the points of departure for the calculation of the Pavement Number:

Rules Relating to the Pavement System In General:

1. The structural capacity of a pavement is a function of (a) the combined long term load spreading potential of the pavement layers; and (b) the relative quality of the subgrade on which the pavement is constructed.

2. The relative quality and stiffness of the subgrade is the departure point for design, as the subgrade is a key determinant in the overall pavement deflection, and in the relative degree of bending and shear that will take place in overlying pavement layers.

3. For pavements with thin surfacings, the base layer is the most critical component, and failure in this layer effectively constitutes pavement failure. Experience can guide the relative confidence in different material types to serve as base layers under heavy traffic.

Rules Relating to Specific Pavement Layers:

4. The load spreading potential of an individual layer is a product of its thickness and its effective long term stiffness under loading.

5. The Effective Long Term Stiffness (ELTS) of a layer depends on the material type and on its situation in the pavement system.

6. Fine-grained subgrade materials act in a stress-softening manner. For these materials, the ELTS is determined mainly by the material quality and by the climatic region. Owing to the stress softening behaviour, subgrade materials will generally soften with decreased cover thickness.

7. Coarse-grained, unbound layers act in a stress-stiffening manner. For these materials, the ELTS is determined mainly by the material quality and the relative stiffness of the supporting layer. The ELTS of these materials will increase with increasing support stiffness, up to a maximum stiffness which is determined mainly by the material quality.

8. Cement stabilized materials initially act as a stiff, glassy material, but gradually deteriorate into a material consisting of loose clumps or separate blocks that can be solid or deteriorated into a granular state. For a specific material class, the rate of deterioration depends mainly on the thickness of the layer and on the stiffness of the support.

9. Thin asphalt surfacings act as either stiff, glassy material, or as semi-stiff, rubbery material. The material state depends primarily on the temperature and binder content. Over time, the material is subject to deterioration owing to ageing and fatigue. Fatigue breakdown is primarily dependent on the stiffness of the supporting layer.

Assumption Relating to the Behaviour of Bitumen Stabilized Materials (see Section 5 for details):

10. Bitumen stabilized materials with low cement contents are assumed to act in a similar way to coarse granular materials, but with a higher cohesive strength. The cohesive strength is subject to breakdown during loading, and thus some softening over time can occur. The rate of softening is mainly determined by the stiffness of the support, which determines the degree of shear in the layer. However, owing to the higher cohesive strength in bituminous stabilized materials, these layers are less sensitive to the support stiffness than unbound granular materials, and thus can sustain higher modular ratio limits.

The above-noted rules-of-thumb introduces several concepts, like the ELTS, Modular Ratio Limit and Stress-stiffening behaviour. In addition, the rules posit a behaviour model for bitumen stabilized materials. These aspects will be discussed in more detail in the following subsections.
3.3 **The Effective Long Term Stiffness (ELTS)**

The ELTS is a model parameter which serves as a relative indicator of the average long term in-situ stiffness of a pavement layer. As such, the ELTS averages out effects of long term decrease of stiffness owing to traffic related deterioration, as well as seasonal variations in stiffness. Thus the ELTS does not represent the stiffness of a material at any specific time.

It is also important to note that the ELTS – as defined for use in the PN - is not a stiffness value that can be determined by means of a laboratory or field test. Rather, it is a *model parameter*, which was calibrated for use in the PN-based design method that was developed as part of this study. The ELTS values used in the calculation of the PN may therefore differ somewhat from the stiffness values that are conventionally adopted for ME design (e.g. the recommended stiffness ranges shown in Theyse et al., 1996).

The ELTS concept is especially needed in the case of cement stabilized materials, where a significant change in the effective stiffness of the material can be expected during the course of a pavement's design life (de Beer, 1990; Theyse et al, 1996; TRH4, 1996). This concept is illustrated in Figure 5, which shows the reported breakdown of a cement stabilized material under traffic, with the ELTS representing an *average effective long term stiffness*.

![Effective Stiffness Chart](image)

**Figure 5: Application of the ELTS Concept for Cement Stabilized Materials**

3.4 **Modelling of Subgrade Materials**

In the adopted model, the first step in the calculation of the PN-value is the determination of the subgrade class. To do this, specific guidelines are provided (see Section 4.4 for details) to consistently determine the placement of the subgrade within the pavement system. A materials classification system was also developed to assist designers to consistently classify the subgrade and other layers using routine pavement condition information. This classification system forms an integral part of the PN-based design method, and is described in detail in a separate report (Jooste, Long and Hefer, 2007).
Once the subgrade class has been determined, the designer can calculate the ELTS for the subgrade. This involves the following steps:

1. Assignment of a basic long term stiffness based on the materials class.
2. Adjustment of the basic long term stiffness for different climatic regions (wet, dry or moderate).
3. Adjustment of the stiffness determined in step 2 to take account of depth of subgrade cover.

The adjustment of the subgrade stiffness to take account of the depth of cover was included in the model to take account of the well-known stress-softening tendencies of fine grained materials, in which these materials tend to soften under load (Huang, 1993; Thompson and Elliot, 1985). In the stress-softening model, the stiffness decreases with increasing deviator stress, and thus for a particular material the relative degree of softening will depend on the depth and stiffness of subgrade cover.

For the adopted PN model, a wide range of pavement structures from the TRH4 design catalogue were analyzed (see Section 6.1 for more detail on this data set). For each structure, the subgrade cover depth was determined and the maximum and minimum limits of subgrade cover depth were then determined. For the upper end of the observed subgrade cover, a fixed increase in the subgrade stiffness was assigned. Conversely, for the lower end of the observed range of cover, a fixed decrease in subgrade stiffness was assigned. For cover depths between the maximum and minimum limits, a linear relationship between subgrade cover and stiffness adjustment was assigned.

Figure 6 shows the subgrade stiffness adjustment for different cover depths. It should be noted that the aim of this adjustment is to mimic subgrade behaviour in a general sense. The maximum and minimum adjustment limits were determined through a calibration process. This process firstly involved an assessment of the general suitability of the adjustment by evaluating the adjusted subgrade values based on experience. A further calibration was then performed to ensure that the adjusted subgrade stiffness resulted in an optimal correlation between the resulting PN-value and observed pavement performance.

If Cover Thickness (in mm) is:
- > 800, then Adjustment = +10 MPa
- < 500, then Adjustment = -10 MPa
- else: Adjustment = -10 + [(Cover-500)/300] * 20 MPa

Figure 6: Assumed Influence of Subgrade Cover Depth on Subgrade Stiffness
3.5 THE MODULAR RATIO LIMIT CONCEPT

The modular ratio is a well known concept in flexible pavement engineering (Maree, 1982), and is defined as the ratio of a layer's stiffness relative to the stiffness of the layer below it. Thus, if the stiffness of a base layer is 300 MPa, and the stiffness of the support below it is 200 MPa, then the modular ratio of the base layer would be 1.5. An analysis of stress-sensitive material behaviour in finite element models show that, as a general rule of thumb, the modular ratio for unbound granular materials is limited to less than 2.5, but in cases of more cohesive materials and weak support, modular ratio's as high as 5.0 may be possible.

The modular ratio concept follows directly from the stress-sensitive stiffness of granular materials, which causes the stiffness of a granular material to decrease when the material is placed over a weaker (less stiff) support. This decrease in stiffness occurs because, in situations where the support layer is soft, the overlying layers tend to bend more into the support, thereby increasing the tendency to develop higher shear and tensile forces in the overlying layers. This effect limits the stiffness that can be obtained in an unbound layer placed over a weaker support.

The impact of the support type on the effective stiffness of an unbound granular material is shown in Figure 7 and Figure 8. These figures show the stiffness and deformation plot for a typical pavement structure, and each figure represents a different subbase situation. In these figures, the values shown at various locations represent the stress-sensitive stiffnesses, in MPa, at that location within the pavement.

Figure 7: Stress-Sensitive Stiffness Distribution (higher quality subbase)

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 mm Medium Density/Quality Crushed Stone (K₁ = 8000, K₂ = 0.67)</td>
<td>94, 93, 93, 93</td>
</tr>
<tr>
<td>300 mm Crusher Run and Gravel Blend (K₁ = 5000, K₂ = 0.5)</td>
<td>93, 93, 93, 93</td>
</tr>
<tr>
<td>2000 mm Stiff Silty-Sandy Clay (stress-softening)</td>
<td>109</td>
</tr>
</tbody>
</table>

Note: numbers denote the stiffness of the material, in MPa, at the location where the number is shown

The stress and stiffness evaluations shown were calculated using the non-linear finite element module of the Rubicon1 software package. The finite element program implements the well-known K1-theta-K2 model for coarse grained (stress-stiffening) materials, and the bilinear stress softening model for fine grained materials such as clay and silt. Details of these models can be found in Huang (1993).
Figure 7 and Figure 8 represent a much more realistic model of the actual pavement behaviour than the traditional linear layered elastic model in which the stiffness of the material in a specific layer is fixed in the vertical and horizontal direction. These figures also exhibit some well-known trends and prompt the following observations:

- The stiffness of a specific layer under load varies from point to point, depending on the stress state at that point. There is thus no such thing as “the stiffness” of a pavement layer. In all cases a stiffness value adopted for a layer, for purposes of design and analysis, is not a fundamental material property but a model parameter, and should be calibrated for use in a particular model. This applies even in the case of the more accurate stress-sensitive finite element model, since smaller or larger elements would have yielded slightly different stiffnesses than those shown here.

- In the case of unbound granular materials, the relative stiffness of the support is a key determinant of the general stiffness of a layer in the area under the load. For the examples shown, the only change is the quality of the subbase material, which was changed from a typical crushed-stone gravel blend (Figure 7) to a gravel sand blend (Figure 8). Taking the average of stiffness of the four elements directly under the load, we see that the base stiffness decreases from 537 MPa to 459 MPa as a result of the weaker subbase.
Figure 9 shows the average base stiffness* of different material types plotted against the average support (i.e. subbase) stiffness. This figure shows the following:

- For the higher quality crushed stone, the layer is highly dependent on the stiffness of the support.
- For the lower quality sand-gravel blend, the support is less important and the intrinsic quality of the material basically determines the stiffness.
- In all cases there is a maximum stiffness that the material can attain. This maximum stiffness is higher for better quality materials.
- The higher quality material can sustain a greater modular ratio. For the examples shown in Figure 9, it can be shown that for a support stiffness of approximately 230 MPa, the high quality crushed stone layer has a stiffness of roughly 756 MPa, which implies a modular ratio of 3.3. By contrast, for similar support stiffness, the medium quality crushed stone can maintain a modular ratio of only 2.4.

Figure 9 also shows that there is a maximum stiffness that each material will be able to develop under loading. As with the modular ratio, the maximum stiffness depends on the quality of the material, and less

* In this context, the average base stiffness refers to the average stiffness of the elements directly under the load. For the base, the value was calculated by averaging the stiffnesses of the four elements directly under the load, and for the subbase, by averaging the stiffnesses of the six elements directly under the load (refer to Figure 7 for the location of these elements).
dense and angular materials can not develop very high stiffnesses under loading, regardless of the stiffness of the support.

It is important to note that the modular ratio that a material can sustain will vary over the life of a pavement. The concept of pavement balance, as discussed by Maree (1982) and Kleyn (1984) essentially assumes that the modular ratio of different unbound layers in a pavement system will decrease over time, as the traffic moulds and densifies the material into a more uniform system.

Thus while it is possible for a high quality crushed stone to maintain a modular ratio of 4 to 5 right after construction, over time the material will be moulded and weakened by traffic into a more balanced state where a modular ratio of 3 or less is likely to be observed. It is thus important to note that the use of a modular ratio limit, as defined here, pertains to the overall long term stiffness that a material can maintain over time.

In the PN based model, the modular ratio limit and the maximum allowable stiffness is used extensively to determine realistic ELTS values. These parameters are used in the following way:

1. The stiffness of the supporting layer is first determined. Thus the PN calculation process starts from the subgrade and proceeds upward toward the surfacing.
2. The modular ratio limit and maximum allowable stiffness is determined based on the material class.
3. The ELTS for a layer is determined as the minimum of (a) the support stiffness multiplied with the modular ratio limit; and (b) the maximum allowed layer stiffness.

In the case of base layers, the ELTS is further adjusted by means of a base confidence factor, which is discussed in more detail in Section 3.8. The use of the modular ratio limit and maximum allowable stiffness is also applied to stabilized materials, as explained in the following subsection.

3.6 MODULAR RATIO LIMIT FOR CEMENT STABILIZED MATERIALS AND HOT MIX ASPHALT

Modular ratio limits, as defined in Section 3.5, normally do not apply to cohesive materials such as cement stabilized layers and hot mix asphalt. This is because of the high cohesion inherent in such materials, which effectively removes the stress-sensitivity and ensures that these materials can maintain a relatively high stiffness under loading, even over weak support.

However, when the long term stiffness of these materials is considered, then the stiffness of the support again becomes relevant. This is because weaker support layers will lead to increased fatigue and hence faster breakdown of stabilized layers. Thus, when these materials are used in a simplified model, the modular ratio limit can serve to mimic the long term fatigue effect that will lead to quicker reduction of the stiffness when these materials are placed over softer support.

This effect is illustrated schematically in Figure 10, which shows a case where the same cement stabilized material is placed over a stiff and soft support. Because the cement stabilized material is the same in both instances, the initial and final stiffness values are the same. However, the material on soft support experiences more rapid stiffness reduction, and thus the effective stiffness over the long term is lower than for the material on the stiff support.

Another factor which determines the rate and degree of breakdown in cement stabilized materials is the thickness of the layer. For example, in the SA Mechanistic Design method (Theyse et al., 1996), a non-linear shift factor is used to adjust the effective fatigue life of cement stabilized materials on the basis of the layer thickness. For example, the effective fatigue life shift factor for a 100 mm thick layer is 1.0, and for a 300 mm layer it is 3.7 (see Figure 6 in Theyse et al., 1996).
To mimic the influence of layer thickness on the rate of deterioration, the PN model adjusts the ELTS of cement stabilized layers for thicknesses below 300 mm. The reduction factor was calibrated using structures from the TRH4 design catalogue and the general relationship for determining this factor is shown in Figure 11.

Figure 10: Modular Ratio Limit For Cement Stabilized Materials

Figure 11: Relationship for Adjusting Cement Stabilized Layer ELTS Based on Thickness

If Thickness (in mm) is:
- > 300, then Adjustment Factor is 1.0
- else:
  Adjustment = 1 - [(300-Thickness)/50] * 0.3
to a minimum of 0.2
(Multiply adjustment factor with layer ELTS)
3.7 **ASSUMED BEHAVIOUR OF BITUMEN STABILIZED LAYERS**

As noted in Section 3.1, the behaviour of bitumen stabilized layers is assumed to be similar to that of unbound granular materials. However, it is also assumed that bitumen stabilized materials are able to develop significantly higher cohesive strength, and thus, compared to unbound granular materials, these materials are less dependent on the stiffness of the support layer.

The assumption of unbound material behaviour with a high cohesive strength places the behaviour of bitumen stabilized layers somewhere between that of a crushed stone and a cement stabilized material. This assumption is supported by observations made from the LTPP sections that were evaluated as part of this study. A detailed discussion of these observations can be found in Section 5.

The assumed behaviour model for bitumen stabilized materials deviates somewhat from the two phase model postulated for foamed bitumen (Asphalt Academy, 2002) and emulsion treated materials (de Beer and Grobler, 1994). In the two phase model, the material is assumed to act first as a stiff material that will fail in effective fatigue. Thereafter, the material is treated as being equivalent to a granular material for which permanent deformation will be the dominant mode of failure.

For the purposes of determining the ELTS value, bitumen stabilized layers are modelled in a similar manner to crushed stone materials. However, in the case of bitumen stabilized layers, a higher modular ratio limit was allowed compared to crushed stone layers, to account for the higher cohesive strength. Calibration confirmed the validity of this model, and showed that the maximum allowable ELTS for bitumen stabilized materials was similar to that of crushed stone (G1 or G2 material).

3.8 **THE BASE CONFIDENCE FACTOR**

An assessment of pavement structures typically used in different traffic scenarios quickly shows that base quality is intimately linked to the intensity of the traffic loading, regardless of the overall pavement structure. For example, for dry regions, the TRH4 design catalogue for unbound base layers shows that only the highest quality crushed stone material (G1) is allowed as a base layer material for design traffic above 10 msa. Similarly, in wet regions, and for Category A and B roads, only a G1 material is allowed as a base layer for design traffic above 1 msa.

A similar trend was observed in the bituminous stabilized pavements, in which only high quality source material (generally graded crushed stone) was used for higher design traffic (see Section 8.4 for details). Thus experience shows that there is a limit on the types of base materials that can be considered for a given traffic situation. In particular, the suitable design options decrease significantly as the design traffic increases.

The failure to explicitly limit the type of material used as a base course is one of the main reasons why index based methods like the AASHTO SN approach are often slighted by engineers. For example, with the traditional AASHTO SN, it is possible to use a poor quality base material in a high traffic situation, and then compensate for this by using a thicker or better subbase. This may increase the SN sufficiently to suit the design requirements, but the method fails to identify a critical weak link in the pavement system.

This situation is specifically relevant in southern Africa, where thin surfaced pavements are the norm. In such situations, the base is the main load bearing element in the pavement system, and failure of the base effectively constitutes pavement failure. In the knowledge based approach, such a situation can be avoided by the application of practical limitations, which are based on experience, and which limit the types of base materials that can be used in certain traffic situations.

* It is important to note that the use of the long term effective stiffness in the PN model to a large extent diminishes the importance of distinct phases (if such phases exist). If a material does have two phase behaviour in which the stiffness and density is significantly reduced under traffic, then this will merely serve to change the ELTS value. It is, however, of interest to note that a distinct deterioration in available layer properties was not noted for any of the LTPP sections investigated. This applies to both bitumen and cement stabilized layers.
In the PN-based method, it is proposed that the appropriateness of the base material be controlled in two ways: (a) a Base Confidence Factor (BCF) should be assigned to different material types; and (b) the design guidelines should include a checklist to ensure that practical considerations, such as the appropriateness of the base material, are taken into account. More details on these practical considerations are provided in Section 8. In the PN model, the BCF thus serves two purposes:

1. **Disqualification of Unpractical Designs:** The BCF allows the PN model to effectively disqualify designs that violate practical considerations, and which have a high likelihood of failing in the base layer owing to crushing or shear deformation. For weak materials, a low or even negative BCF is assigned. This factor would thus significantly reduce the PN for unpractical design solutions (e.g. where a weak sand-gravel blend is used as a base layer in a medium to high traffic scenario).

2. **Reflecting Proven Capability of Certain Base Materials:** The BCF is used to reflect the trend, observed in the knowledge base (particularly the knowledge base inherent in the TRH4 design catalogue), in which certain material types are not used above a certain design traffic limit, depending on the climate. The BCF was calibrated for different materials to allow the PN model to accurately reflect this trend.

In the PN model, the BCF is used to adjust the ELTS value for the base layer. This is done simply by multiplying the initial ELTS for the base with the BCF.

### 3.9 SUMMARY

This section outlined the knowledge base used to develop a PN that effectively incorporates the main principles of pavement behaviour and performance. The rules-of-thumb which were used as points of departure for the PN model were discussed, as were the key concepts needed to quantify these rules in a heuristically determined PN. These concepts can be summarized as:

- In the PN model, the effective long term load spreading capacity of a layer is represented by a model parameter known as the effective long term stiffness (ELTS).

- The calculation of the ELTS for a specific layer depends on the material type and on the situation in which the layer is placed. The stiffness of the supporting layer is of particular importance in determining the ELTS.

- The ELTS of the subgrade is the starting point for design, and greatly influences the relative stiffness of the overlying pavement system. In the PN model, the ELTS of the subgrade is determined by the material class, the climate and by the depth of cover over the subgrade.

- The general method for determining the ELTS relies on the modular ratio limit and the maximum allowable stiffness. For these parameters, different values are assigned to different material types and were calibrated using the available knowledge base of pavement structural capacity.

- The ELTS of a layer is determined as the minimum of (a) the support stiffness multiplied by the material’s modular ratio limit; and (b) the maximum allowable stiffness assigned to the material type.

- The modular ratio concept is also applied to highly cohesive materials such as cement stabilized materials and hot mix asphalt. In these cases, the modular ratio serves to quantify the influence of support stiffness in determining the rate at which such materials will break down due to fatigue.

- In the case of cement stabilized materials, the layer thickness is also used to adjust the ELTS. This adjustment mimics the effect that layer thickness has on the rate of crack propagation in these materials.

- Bitumen stabilized materials are assumed to behave in a manner similar to crushed stone material, but with a higher cohesive strength, which is simulated by the assignment of a higher modular ratio limit.

- A Base Confidence Factor is incorporated in the PN model to ensure that inappropriate base types are not used in high traffic scenarios.
4 CALCULATION OF THE PAVEMENT NUMBER

In the previous section the basic concepts underlying a heuristically determined Pavement Number (PN) were presented. The discussion introduced the main concepts involved in the model, which included the ELTS, modular ratio limit, maximum allowable stiffness and base confidence factor.

In this section, the method for calculating the PN is detailed. The discussion presents a step-by-step outline of the calculation process and, where needed, clarifies concepts related to specific steps.

4.1 METHOD FOR DETERMINING THE PN

A stepwise outline of the method for determining the pavement number is provided below. Details relating to different steps or concepts in the method are discussed in the subsections that follow.

Step 1: Check to ensure that the design method is applicable for the design situation (see Section 4.2 for details). If the design method is not applicable, a more detailed analysis should be performed, and the PN based method should not be used.

Step 2: Determine the layer thicknesses, and available material properties for each layer. Use the material properties to obtain a design equivalent material class for each layer (see Section 4.3 for details).

Step 3: Combine layers with similar properties to obtain a five layer pavement system, including the subgrade (see Section 4.4 for details).

Step 4: Determine the basic stiffness of the subgrade by means of Table 3. Adjust the stiffness for climatic region (Table 4) and depth of subgrade cover (Figure 12) by multiplying the basic stiffness by the adjustment factors. The resulting stiffness is the ELTS for the subgrade.

Step 5: For each layer above the subgrade, determine the modular ratio limit and maximum allowed stiffness from Table 5.

Step 6: Use the modular ratio limit and maximum allowable stiffness to determine the ELTS for each layer by working up from the subgrade (see Section 4.5 for details).

Step 7: For the base layer, determine the Base Confidence Factor (BCF) from Table 5, and for cement stabilized layers, determine the adjustment factor based on thickness from Figure 13.

Step 8: For each layer, calculate the layer contribution by multiplying the ELTS with the layer thickness and dividing this by 10 000. For the base layer multiply this product with the BCF, and for any cement stabilized layers, multiply with the thickness adjustment factor.

Step 9: Add the layer contributions for each layer to get the PN.

It should be noted that the constants shown in Table 3 to Table 5 were obtained through an iterative calibration process. The values shown are those which strike the best balance between (a) providing a good agreement with the knowledge base of observed performance; and (b) the engineering judgement and experience of the development team. This calibration process is described in detail in Section 4. Since the model constants were calibrated for a specific knowledge base, they should not be adjusted by the designer.

* In a normal pavement design situation, these steps will need to be applied for each uniform design section. For rehabilitation design situations, it is thus presumed that the designer will have detailed information on the existing pavement layer properties within the design section.
### Table 3: Stiffness Determination for the Subgrade

<table>
<thead>
<tr>
<th>Design Equivalent Material Class for Subgrade</th>
<th>Stiffness Value (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G6 or better</td>
<td>180</td>
</tr>
<tr>
<td>G7</td>
<td>140</td>
</tr>
<tr>
<td>G8</td>
<td>100</td>
</tr>
<tr>
<td>G9</td>
<td>90</td>
</tr>
<tr>
<td>G10</td>
<td>70</td>
</tr>
</tbody>
</table>

**Note:** Subgrade stiffness value should be adjusted for climate and cover depth

### Table 4: Climate Adjustment Factors

<table>
<thead>
<tr>
<th>Climate and Weinert N Values (after TRH4, 1996)</th>
<th>Adjustment Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet (Weinert N &lt; 2)</td>
<td>0.6</td>
</tr>
<tr>
<td>Moderate (Weinert N = 2 to 5)</td>
<td>0.9</td>
</tr>
<tr>
<td>Dry (Weinert N &gt; 5)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

### Table 5: Modular Ratio Limit and Maximum Allowed Stiffness for Pavement Layers

<table>
<thead>
<tr>
<th>General Material Description</th>
<th>Design Equivalent Material Class</th>
<th>Modular Ratio Limit</th>
<th>Maximum Allowed Stiffness (MPa)</th>
<th>Base Confidence Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot mix asphalt (HMA) surfacing and base material</td>
<td>AG, AC, AS, AO</td>
<td>5.0</td>
<td>3500</td>
<td>1.0</td>
</tr>
<tr>
<td>Surface seals</td>
<td>S1, S2, S3, S4, S5, S6</td>
<td>2.0</td>
<td>800</td>
<td>N/A</td>
</tr>
<tr>
<td>High strength bitumen stabilized material, normally using crushed stone or reclaimed asphalt pavement (RAP) source material</td>
<td>BSM1</td>
<td>3.0</td>
<td>600</td>
<td>1.0</td>
</tr>
<tr>
<td>Medium strength bitumen stabilized material, normally using natural gravel or RAP source material</td>
<td>BSM2</td>
<td>2.0</td>
<td>450</td>
<td>0.7</td>
</tr>
<tr>
<td>Crushed stone material</td>
<td>G1</td>
<td>2.0</td>
<td>700</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>G2</td>
<td>1.9</td>
<td>500</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>G3</td>
<td>1.8</td>
<td>400</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>G4</td>
<td>1.8</td>
<td>375</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>G5</td>
<td>1.8</td>
<td>320</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>G6</td>
<td>1.8</td>
<td>180</td>
<td>-2.0</td>
</tr>
<tr>
<td>Natural Gravel</td>
<td>G7</td>
<td>1.7</td>
<td>140</td>
<td>-2.5</td>
</tr>
<tr>
<td></td>
<td>G8</td>
<td>1.6</td>
<td>100</td>
<td>-3.0</td>
</tr>
<tr>
<td></td>
<td>G9</td>
<td>1.4</td>
<td>90</td>
<td>-4.0</td>
</tr>
<tr>
<td></td>
<td>G10</td>
<td>1.2</td>
<td>70</td>
<td>-5.0</td>
</tr>
<tr>
<td>Gravel-soil blend</td>
<td>C1 and C2</td>
<td>9</td>
<td>1500</td>
<td>0.8</td>
</tr>
<tr>
<td>Cement stabilized crushed stone</td>
<td>C3</td>
<td>4</td>
<td>550</td>
<td>0.6</td>
</tr>
<tr>
<td>Cement stabilized natural gravel</td>
<td>C4</td>
<td>3</td>
<td>400</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Calculation of the Pavement Number

Figure 12: Adjustment of Subgrade Stiffness Based on Cover Thickness

Figure 13: ELTS Adjustment Factor for Cement Stabilized Layers based on Layer Thickness
4.2 **Suitability of the PN-Based Design Method**

Before the calculation of the pavement number is started, the designer should check to ensure that the design method is applicable to the pavement situation. To ensure that the method is not used inappropriately, the designer should always check to ensure that none of the following situations apply:

1. **Design traffic greater than 30 msa:** The PN-based method was calibrated using a knowledge base which was limited to pavements that had accommodated less than 30 million standard axles (msa). Thus, in such a design situation, the design must be checked using a more in-depth mechanistic-empirical analysis, in which the stresses and strains are evaluated by means of calibrated transfer functions.

2. **Presence of thin, weak lenses:** If thin, weak lenses of material exist below the surfacing, or between stabilized layers, then zones of high slip and shear will develop, and routine design calculations will not apply. In such instances, the structural capacity assessment of the PN-based method, or of the traditional ME design method will not be appropriate, and special treatment of the affected weak lens must be undertaken. The PN-based design method cannot be applied to situations where such lenses still exist within the pavement structure, especially where such lenses are located within the upper 400 mm of the pavement structure.

3. **Design traffic less than 1 msa:** in cases where the design traffic is less than 1 msa, the designer should use a design catalogue. It is proposed that the design catalogue be developed once pilot testing of this proposed method is completed.

4. **Subgrade CBR less than 3 per cent:** the knowledge base on which the PN based method was calibrated did not include any pavements in which the subgrade CBR was less than 3 per cent. The PN based method should therefore not be used in cases where the subgrade CBR is less than 3 per cent at a depth below 600 mm.

4.3 **Material Classification**

The derivation of the material classes for each pavement layer is a critical aspect of the design process, since – in this method – this process effectively constitutes the determination of design inputs. A detailed methodology was therefore developed to determine the material design classes either from specifications or from available material and pavement test indicators. This process is described in detail in Jooste et al (2007), and only a brief overview of the classification method will be provided here.

The material classification method strives to provide designers with a consistent and rational method for interpreting a variety of material and pavement test indicators obtained within a uniform subsection. The test indicators can range from relatively sophisticated (e.g. shear strength parameters obtained from triaxial testing) to simple indicators (e.g. DCP penetration rate, Plasticity Index, etc.). The indicators can also be tested at different sampling frequencies, which typically results in different sample sizes for different indicators.

For example, for a given uniform subsection, a designer may have test data from two test pits (with associated test indicators such as grading, moisture content, Plasticity Index, etc.), ten DCP tests (yielding the average penetration rate per layer), and 50 Falling Weight Deflectometer test points (yielding estimated layer stiffnesses under FWD loading). The materials classification method provides a designer with fixed guidelines to interpret different test indicators, and a method to synthesize all available indicators to obtain an overall weighted indicator of the relative quality of each layer (expressed as a material class).

The guidelines for interpreting test indicators are based on established pavement material knowledge, and assists designers to interpret information such as: material grading, fines content, Plasticity Index (PI), moisture content, backcalculated stiffness, DCP penetration rate, etc. The guidelines contain tables and graphs that allow designers to quantify such parameters in terms of the likely shear strength of the material (Jooste et al., 2007).
The method used to synthesize the quantified interpretation of available test data is based on concepts of Certainty Theory (Hopgood, 2001; Shortliffe and Buchanan, 1975) and Fuzzy Logic (Hopgood, 2001; Zadeh, 1975). The method facilitates the synthesis of different sources of information (called “evidence”) to obtain a likely material classification. In this synthesis process, the method quantified two sources of uncertainty, namely: (i) the uncertainty introduced by weak interactions between shear strength and individual indicators such as DCP penetration, PI and so forth; and (ii) uncertainty caused by the use of a sampling estimate, often using very small sample sizes.

As noted earlier, the materials classification method is discussed in detail in Jooste et al. (2007). If this process is followed, the designer will obtain a consistent estimate of the material class for each pavement layer. For the PN-based design system proposed here, these material classes, together with the layer thicknesses, effectively constitute the inputs for structural design. For layers that are planned to be recycled using bitumen stabilization, the designer will need to assume different material classes to determine which class will provide the optimal balance of cost, ease of construction and structural capacity.

One aspect that deserves further discussion here is the material classes used in the classification system, specifically in a rehabilitation design context. For new pavements, or layers that are to be reworked or added to the current pavement, it seems natural to adopt the current classification system for materials as defined in the TRH14 document, which provides guidelines for road construction materials (TRH14, 1985). However, for existing pavement layers that will remain undisturbed in the rehabilitated pavement system, the concept of materials classification is more complex.

For example, consider an overlay design on a uniform subsection in which three test pits were opened. If two of the three test pits indicate the base is a G2 quality material, and the third test suggests the material is a lesser quality G3, which material class should be assumed for design? This question can perhaps be solved in a statistical sense, but conceptually it is more difficult to interpret. After all, considering the subsection as a unit, the base layer is neither a G2 nor a G3, but some combination of the two. This becomes even more complex if the pavement is showing base-related deterioration such as pumping and shallow rutting, since the layer is no longer equivalent to a newly constructed G2 or G3.

In a rehabilitation context, a broader classification system should perhaps be adopted, using, for example, classes such as “high quality crushed stone”. However, the use of such classes would mean that in many instances different quality materials will be grouped together, thereby decreasing the precision of the design method. Another approach would be to introduce a new materials classification system. However, this is likely to create some confusion and resistance to implementation.

After consideration of the problem, it was decided to adopt the existing TRH14 classification system, but with understanding that the obtained materials class will be regarded as the design equivalent materials class. So, 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. The materials classification system described in Jooste et al (2007) would then provide a consistent method that will document the necessary evidence to support this classification.

4.4 COMBINING PAVEMENT LAYERS TO FORM A FIVE LAYER MODEL

By definition, the PN consists of the sum of the load spreading contributions of four pavement layers above the subgrade. To apply this definition consistently, the pavement model used in the PN calculation must consist of four pavement layers plus the subgrade. In cases where the pavement consists of more than four layers, two or more layers will need to be combined. To do this, the following guidelines should be adhered to:

* For brevity, the term materials class will generally be used in this document. However, the reader should bear in mind that – in the case of layers in an existing pavement structure – the assigned material class always denotes the design equivalent materials class.
• Only combine layers that consist of the same general materials class. In this respect, the following general material classes can be used: (a) crushed stone material; (b) natural gravel; (c) cement stabilized material; (d) cement or bitumen stabilized material; and (e) gravel-soil, silt or clay materials.

• The surfacing should be modelled as a separate layer in all cases. A surface seal should be modelled as a 5 mm thick layer.

• Where there is a need to combine pavement layers, the designer should first combine sub-layers below the subbase, followed (if needed) by sub-layers in the subbase zone.

• The material class assigned to the combined layer should be the class of the thicker of the two layers. Thus, if a 120 mm G6 is combined with a 150 mm G7, then the material class assigned to the combined layer should be G7.

• Where the two layers to be combined are of equal thickness, the lower material class should be assigned to the combined layer. Thus, if a 150 mm G7 is combined with a 150 mm G8, then the material class assigned to the combined layer should be G8.

• When a pavement layer is combined with the apparent natural subgrade, the material class of the combined subgrade layer should be the class of the uppermost layer.

• When a pavement consists of only two or three pavement layers, a four layer pavement system should be constructed by subdividing the top of the subgrade into two or more layers, each with a thickness of 100 mm. The material class assigned to these sub-layers should be that of the subgrade.

In most pavement design methods, there is the danger that inexperienced designers may try to increase the apparent structural capacity of the pavement by assuming inappropriate inputs. In the PN based method, this danger is to a large extent avoided by the use of material classes as design inputs. Since material classes are either coupled to available test evidence (as in the case of rehabilitation design) or to material specifications (in the case of newly added layers), designers cannot manipulate material classes without violating the link to available evidence or specifications.

However, there is still the danger that inexperienced designers may try to manipulate the apparent structural capacity by assuming impractical layer thicknesses, specifically for the less expensive unbound layers. For example, a designer can increase the apparent design capacity by assuming a 500 mm thick gravel-sand subbase. Because of the relatively high thickness, this will lead to an increase in the PN, and thus also in the apparent structural capacity. In practice, however, such a thick layer will tend to deform and compact under traffic, and to some extent the layer will effectively form the upper subgrade.

Closely linked to this issue is the difficulty of determining the exact location of the subgrade within a pavement structure, especially when the subgrade consists of a coarse material with a relatively high strength. It also occurs when there are many layers of similar (and relatively poor) quality within the pavement structure.

To address the issues related to unrealistic layer thickness assumptions, and the location of the subgrade, a maximum and minimum practical design thickness are prescribed for different material types. These layer thicknesses are shown in Table 6. In some cases where existing layers are combined to form a five layer structure, the thickness of combined layers may be more than the prescribed maximum thickness in Table 6. In such situations the maximum allowable thickness from Table 6 should be used for the purposes of calculating the PN.

Figure 14 shows an example in which there are several selected layers that are combined to form a five layer system. This example also shows the application of the limiting thickness to the selected layers, which also determines the position of the subgrade for modelling purposes. It should be noted that, even with the above noted guidelines taken into account, some pavement situations will allow more than one approach to the combination of layers. In such cases, the designer should experiment with different approaches and adopt the most conservative model for design purposes.
Table 6: Recommended Layer Thickness Limits for Design Calculations

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Layer Situation</th>
<th>Thickness Limits Allowed for PN Calculation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Hot Mix Asphalt Surfacing Layers</td>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>Surface Seals Surfacing Layers</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Bitumen Stabilized Layers Base and Subbase</td>
<td>100</td>
<td>350</td>
</tr>
<tr>
<td>Cement Stabilized Layers Subbase</td>
<td>100</td>
<td>400</td>
</tr>
<tr>
<td>All Unbound Materials (G1 to G10) Subbase</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>Selected Layer(s)</td>
<td>100</td>
<td>300</td>
</tr>
</tbody>
</table>

Figure 14: Example of the Combining of Pavement Layers to Form a Five Layer Model
4.5 Determining Effective Long-Term Stiffness (ELTS) Values

The concepts of the ELTS and modular ratio limit were discussed in Section 3. In the case of the subgrade, the designer first determines the ELTS using the material class. This value is then adjusted for climate and for depth of subgrade cover.

The climate adjustment of the subgrade stiffness takes into account the increased frequency and risk of having a soft subgrade in wet regions. Climate adjustment factors are shown in Table 4. The adjustment for subgrade cover takes into account the well known behaviour of finer-grained materials (Huang, 1993; Thompson and Elliot, 1985) in which such materials tend to soften under increased stress. A relative adjustment (decrease) of the subgrade stiffness is therefore made to simulate the effect of stress-softening for pavements with less subgrade cover (i.e. where shear stresses are greater). The adjustment of the subgrade stiffness for depth of cover is shown in Figure 12. As with the modular ratio limits and ELTS values, this adjustment is model specific and was determined by calibration against the pavement performance knowledge base.

For pavement layers above the subgrade, the designer can use Table 5 to look up the maximum allowed stiffness and modular ratio limit for each material. The stiffness can then be determined by working from the subgrade upwards, using the modular ratio limit and the maximum allowable stiffness. The assigned ELTS is determined as the minimum of: (a) the maximum allowed stiffness; and (b) the stiffness of the support layer multiplied by the modular ratio limit. In the case of the base layer, the ELTS is also adjusted by multiplication of the ELTS with the base confidence factor.

An example of this procedure is shown in Figure 15 and Figure 16 for a structure in a wet and dry climate, respectively. A comparison of these figures shows the impact of climate on the subgrade, and the subsequent impact on pavement layer stiffnesses. These examples show how the modular ratio effectively takes into account the stiffness of the support layer, thereby reducing the assigned ELTS when the support stiffness reduces, even though the material classes remain unchanged.

![Diagram showing ELTS determination process](image)

**Figure 15:** Example of ELTS Determination (Wet Climate)
4.6 EXAMPLE CALCULATION

An example of the PN calculation is shown in Figure 17. For this example, the following pavement structure was assumed, for a moderate climate:

- 30 mm Asphalt Surfacing
- 175 mm BSM2 (Bitumen Stabilized Natural Gravel)
- 200 mm G6 (Gravel Soil Blend)
- 180 mm G7 Selected Layer
- G8 Subgrade

The first steps involved the determination of the subgrade ELTS, and is shown in the topmost table in Figure 17. To calculate this value, the initial stiffness was first determined from Table 3, and then adjusted for climate (using Table 4) and cover depth (using Figure 12). The subgrade ELTS is then entered into the last row of column 4 of the lower table in Figure 17.

The next step involves the ELTS calculation for each layer. First, the modular ratio limit and maximum allowed stiffness is determined from Table 5. The ELTS is then calculated as the minimum of: (a) the maximum allowed stiffness; and (b) the stiffness of the support layer multiplied by the modular ratio limit. Since the support stiffness is needed to calculate the ELTS, the calculation starts at the subgrade and then moves upward.

Once the ELTS for each layer is determined, the thickness adjustment factor is determined from Figure 13 and entered in column 5. This factor applies only to cement stabilized materials and is therefore shown as 1.0 for all layers in this example. The BCF for the base layer is then determined from Table 5.

The layer contribution in column 7 is calculated by multiplying for each layer, the thickness with the ELTS and thickness adjustment factor. This product is then divided by 10,000 to scale the number to a realistic value. For the base, the product is also multiplied with the BCF.

Figure 16: Example of ELTS Determination (Dry Climate)
The layer contributions are then added to provide the PN. It can be seen from Figure 17 that the calculation can easily be programmed into a spreadsheet, and constants and adjustment factors can be determined using a lookup function.

<table>
<thead>
<tr>
<th>Subgrade Class</th>
<th>G8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Stiffness</td>
<td>100 (From Table 3)</td>
</tr>
<tr>
<td>Climate</td>
<td>Moderate</td>
</tr>
<tr>
<td>Climate Adjustment</td>
<td>0.9 (From Table 4)</td>
</tr>
<tr>
<td>Cover Depth</td>
<td>585</td>
</tr>
<tr>
<td>Cover Adjustment</td>
<td>-4 (From Figure 12)</td>
</tr>
<tr>
<td>Subgrade ELTS</td>
<td>86</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (mm)</th>
<th>Material Class</th>
<th>Modular Ratio (Table 5)</th>
<th>Max Emod (Mpa) (Table 5)</th>
<th>ELTS (Mpa)</th>
<th>Thickness Adjustment</th>
<th>BCF</th>
<th>Layer Contribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surfacing</td>
<td>30</td>
<td>AC</td>
<td>5.0</td>
<td>3500</td>
<td>1800</td>
<td>1.0</td>
<td>N/A</td>
<td>5.4</td>
</tr>
<tr>
<td>Base</td>
<td>175</td>
<td>BSM2</td>
<td>2.0</td>
<td>150</td>
<td>360</td>
<td>1.0</td>
<td>0.7</td>
<td>4.4</td>
</tr>
<tr>
<td>Subbase</td>
<td>200</td>
<td>G6</td>
<td>1.8</td>
<td>180</td>
<td>180</td>
<td>1.0</td>
<td>N/A</td>
<td>3.6</td>
</tr>
<tr>
<td>Selected</td>
<td>180</td>
<td>G7</td>
<td>1.7</td>
<td>140</td>
<td>140</td>
<td>1.0</td>
<td>N/A</td>
<td>2.5</td>
</tr>
<tr>
<td>Subgrade</td>
<td>N/A</td>
<td>G8</td>
<td>N/A</td>
<td>N/A</td>
<td>86</td>
<td>Pavement Number = 16</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 17: Example Showing Determination of the Pavement Number

4.7 SUMMARY

This section described the proposed method for calculating the PN of a pavement structure. This method essentially implements the basic principles of pavement behaviour and performance outlined in Section 3. The constants required by the model were discussed and the use thereof illustrated by means of a worked example. These constants were obtained through a systematic calibration with observed pavement capacity, a process which is described in detail in Section 6. Once a designer has calculated the PN, the value can be used to determine the appropriate design capacity for the structure. The process of deriving design criteria for the PN is described in Section 7.
5 OBSERVATIONS ON THE LONG TERM BEHAVIOUR OF BITUMEN STABILIZED MATERIALS

The collection of detailed information on the long term behaviour and performance of pavements of bitumen stabilized layers formed a significant component of this study. Overall, more than 120 pavements involving bitumen stabilization were identified. Of these, 30 pavements proved to have enough available details to enable a study of the long term field performance.

Details of the data collection process, together with summaries of the Long Term Pavement Performance (LTPP) of each of these sections can be found in Long and Jooste (2007). The LTPP section summaries contain details of the construction, behaviour indicators, performance indicators and an analysis of the traffic accommodated by each LTPP section.

This section will summarize key observations following from an analysis of LTPP section behaviour. The objective of the discussion is to provide a basis from which a behaviour model for bitumen stabilized materials can be constructed. In particular, the discussion aims to clarify whether bitumen stabilized materials – as commonly used in southern Africa – act primarily as stiff, brittle material, or as unbound, stress-sensitive granular material.

The discussion firstly focuses on the observations drawn from the LTPP section summaries that can be found in Long and Jooste (2007), after which a basic behaviour model for bitumen stabilized materials is postulated.

5.1 OBSERVATIONS ON LTPP SECTION BEHAVIOUR

The following paragraphs summarize key observations extracted from the LTPP summaries for pavements with bitumen stabilized layers. In general, the information available for the assessment of layer behaviour was limited to surface deflections, DCP penetration values, and – in selected cases – material strength and classification data.

The discussions that follow focus only on the behaviour of the bitumen stabilized layers for those sections that provided useful information. Very limited construction and historical details are provided here, as full details can be found in the complete section summaries (Long and Jooste, 2007). A brief summary of the pavement configuration of each section can also be found in Table 8 of Section 6.

N1 Section 1 near Kraaifontein

This section was rehabilitated in 1984, after the road had been in service for approximately 14 years. The rehabilitation consisted of a reworking and emulsion stabilization of the upper half of a 200 mm thick cement stabilized layer. DCP tests performed in 1989 showed a penetration rate of roughly 1 mm/blow in the emulsion treated layer, while the cement stabilized support could not be penetrated. A research investigation undertaken in 1997 found that intact cores could not be extracted from the emulsion treated layer. Deflections taken at various stages (and with various devices) between 1989 and 2005 do not show a clear trend, and show at most an increase of 200 micron over a 18 year service life in which an estimated traffic loading of 12 to 16 million equivalent standard axles (mesa) traffic had been accommodated.

The data available on N1 Section 1 suggest that the emulsion treated layer acted more like a relatively stiff but unbound layer than a stiff, bound material. Key to this interpretation is the fact that the emulsion treated layer could be penetrated with the DCP (even though the penetration rate was low). By contrast,

* The available deflections on LTPP sections are difficult to interpret, since the measurement devices and sample sizes vary over the analysis period. In some instances the reported averages included both directions, and in other cases deflections are reported separately for different directions. The discussion of deflection trends given here is thus deliberately kept fairly general.
the cement stabilized subbase, which by that stage had accommodated 12 years of pre-rehabilitation traffic, construction traffic including heavy compaction equipment, plus another five years of post-rehabilitation traffic, could not be penetrated. This clearly distinguishes the stiff, glassy behaviour of the cement stabilized layer from the more flexible emulsion treated layer.

N1 Sections 13 and 14 (Springfontein to Trompsburg)
These sections were rehabilitated around 1980, after the road had been in service for roughly 10 years. The rehabilitation consisted of a reworking and emulsion stabilization of the existing cement stabilized base. DCP tests performed in 1994 showed that the emulsion treated layer could be penetrated, and this layer was generally classified as a G5 or G6 material, based on DCP data and test pit findings.

Deflections taken at various stages between 1994 and 2006 do not show a clear trend, and although deflections appear to have increased at some stages, the difference between the average deflections taken in 2006 and 1994 appear to be less than 50 micron. Given the impact of seasonal variations on deflections, this does not seem to indicate a significant breakdown or phase change in any pavement layer. The deflections recorded at various stages are all relatively high with the average deflection varying from roughly 560 to 860 micron. This suggests a fairly flexible pavement at most stages of the pavement life after 1994. It is believed that the data available on N1 Section 13 and 14 suggest that the emulsion treated layer behaved more like an unbound layer than a stiff, brittle material.

N7 Sections 7 (Garies to Okiep)
This section was rehabilitated around 1987. In this construction, it appears that the top 150 mm of an existing 200 mm cement treated subbase was reworked and stabilized with emulsion. DCP tests conducted in 1989 showed that the emulsion treated base could be penetrated, but with difficulty and the test had to be halted after 100 mm had been penetrated. This suggests that the remaining part of the old cement treated subbase was still intact and could not be penetrated. Additional DCP tests conducted in 1997 showed an average penetration rate of 1.1 mm per blow in the emulsion treated layer.

This road represented one of two (out of eight) roads in which intact cores could be extracted in a 1997 research study on emulsion treated roads. On this road, two cores were extracted and these showed an average UCS of 10.2 MPa, with a standard deviation of 6.8 MPa, which suggests a material with the crushing strength of a highly cemented material. These results do not correspond with the fact that the layer could be penetrated by the DCP, and suggests that the cores that were tested represented the cemented subbase rather than the emulsion treated base. Deflections taken at various stages between 1989 and 2005 remain very stable, and do not show any significant increase to suggest a significant breakdown or phase-change in any layer.

MR27 (near Stellenbosch)
This section was rehabilitated around 1988. In this construction, the top 100 mm of an existing 200 mm cement treated subbase was reworked and stabilized with emulsion. DCP tests conducted in 1989 showed that the emulsion treated base could be penetrated, but with a low average penetration rate of 1 mm per blow. In 8 out of 9 tests, the DCP failed to penetrate at a depth that corresponds roughly with the depth of the old existing cemented subbase. This suggests again that the newly constructed emulsion treated layer was in a more granular, elastic state than the cement stabilized subbase.

A 1997 investigation showed that intact cores could not be extracted from the emulsion treated layer. Deflections do not show any clear trend over roughly 12 years of service, and if anything appear to have decreased.

MR504 (near Shongweni)
This section was rehabilitated around 1995 and involved foamed bitumen treatment using either reclaimed asphalt pavement (RAP) or weathered granite. Three different subsections (designated A, B and C) were delineated depending on the source material and construction technique. There appears to be a significant difference between the behaviour and perhaps also the performance of Section A, in which RAP was used, and Sections B and C, in which weathered granite was used.

Section C appears to be the weakest section, as the design thickness of the foamed bitumen layer on this section was the lowest (150 mm as opposed to 175 mm for Sections A and B). Also, the selected layer appears to have incorporated in-situ material, whereas for Sections A and B the selected layer was imported G6 material.
On this project, intact cores could be extracted after roughly 10 years in service. The resilient moduli from the cores were consistently higher than 1500 MPa, which is similar to that of asphalt. Although the UCS values tested on the RAP section (Section A) were significantly higher than on sections B and C, all values suggest a consistency similar to a cemented stabilized layer (C3). The ITS values tested on the RAP section varied roughly from 600 to 1000 kPa, while the ITS tested on the weathered granite section was between 200 and 400 kPa. Clearly, the nature of the source material and binder content has a significant impact on the behaviour of the material over the long term.

In the case of the RAP material, Benkelman beam deflections measured in 1996 and 2004 showed a definite decrease. The weathered granite sections, on the other hand, showed a definite increase in deflections.

The behaviour of these foamed bitumen sections clearly exhibit a different pattern compared to most of the emulsion treated sections discussed earlier. These sections also shows the importance of the source material in determining layer behaviour. This aspect can be accounted for in the mix design stage, and even simple design tests such as the dry and soaked stability test showed significant differences between the stability of foamed bitumen made up with RAP and weathered granite.

The behaviour of these foamed bitumen sections suggest a greater stiffness and perhaps a more cohesive behaviour compared to the emulsion treated sections. However, a visual inspection of the road in 2006 by one of the authors showed no block cracking as would normally be associated with cement stabilized material.

N2 Section 16 (near Kwelera, East London)

This section was rehabilitated around 1981. Rehabilitation involved the cement and lime stabilization of the original base, coupled with the addition of a new emulsion treated base. DCP tests performed in 1989 showed low penetration rates in the emulsion treated base while the cement and lime stabilized subbase could not be penetrated in most of the tests. Additional DCP tests performed in 1997 showed that the base could not be penetrated. The 1997 investigation failed to extract intact cores from the section.

Deflections indicate two uniform subsections exist within this LTPP section, one with a much higher deflection (greater than 600 micron) and the other with a lower deflection (less than 300 micron). This deflection pattern was observed in deflections taken throughout the life of the pavement (1989 to 2005), and a significant increase in deflections is not apparent.

Test pits opened in 2002 recorded the base as a granular material that conforms to the grading of a good crushed stone (G2). It is interesting to note that the stabilized subbase was recorded as an “intact sandy gravel imported slightly stabilized and unstabilized”. Again, the emulsion treated layer seems to conform more to the behaviour of an unbound layer, while the cement treatment in the subbase is still apparent after more than 20 years in service.

N3 Section 4 (near Mooi River)

This section was rehabilitated around 1988. Rehabilitation involved milling and emulsion treatment of existing cement stabilized layers. The rehabilitated pavement had a 200 mm emulsion treated base over a 150 mm cement stabilized crushed stone layer. DCP tests performed in 1990 showed low penetration rates in the emulsion treated base while the cement stabilized subbase could not be penetrated.

This pavement represents one of two (out of eight) emulsion treated pavements from which intact cores could be extracted as part of a 1997 research study. The average UCS tested on two cores was 3 MPa, with a standard deviation of 1 MPa. This suggests a crushing strength similar to a cement stabilized (C3) material, which is surprising, since DCP tests performed as part of the same study showed an average penetration rate within the emulsion treated base of 2.6 mm per blow.

FWD deflections measured in 2004 suggest a very stiff pavement, and no decrease in deflections between 1990 and 2004 is apparent.

Other LTPP Sections

The above paragraphs summarize observations on sections which contained relatively comprehensive indications of pavement behaviour. Most of the other LTPP sections did not contain any behaviour indicators apart from deflections. A general assessment of available deflections show that on no section did a significant increase in deflections occur over the monitoring period.
5.2 DISCUSSION

The observations noted on the LTPP sections suggest that bitumen stabilized materials (and emulsion treated materials in particular), do not exhibit the same behaviour as cement treated materials. The DCP tests, in particular, suggest that cement stabilized layers retain a considerable hardness, even after a significant amount of trafficking. The bitumen stabilized layers, on the other hand, tend to conform more to the behaviour of an unbound, dense granular material.

In most instances, the bitumen stabilized layers could be penetrated with the DCP, and exhibited penetration rates roughly between 1 and 3 mm. The cement stabilized layers could generally not be penetrated, even when these layers had been in service much longer than the bitumen stabilized layers.

A distinct change in the behaviour of any pavement layers, or of the pavement system as a whole, could not be observed in any of the LTPP sections. Sections which exhibited high deflections soon after construction also did so after many years in service, and sections with low deflections displayed similar deflections after many years in service.

The attempt to extract cores from eight emulsion treated pavements in 1997 showed that the emulsion treated layers were generally in an unbound, granular state, as intact cores could only be extracted on two out of the eight roads. It is, however, important to note that for those cases where cores could be extracted, the compressive strength of the material differed significantly. Furthermore, the behaviour of two foamed bitumen sections suggests that bitumen treated materials may develop and retain enough cohesive strength to ensure that the material can be cored. Clearly, the binder content will play a crucial role in determining whether the material is primarily in a bound or unbound state.

It is thus very difficult to confidently postulate a general behaviour model that will apply for all bitumen stabilized materials. The nature and stiffness of these materials is likely to vary significantly depending on the bitumen and cement content used. The cement content, in particular, is a key determinant of the behaviour and performance of bitumen stabilized materials. Indirect Tensile Strength (ITS) tests reported by Long and Theyse (2001) suggested that the material behaviour becomes dominated by the cement content if excessive cement is used. In these cases, the benefit of added bitumen is probably negligible.

Discussions with various practitioners during the course of this study showed some disagreement over the role of cement in bitumen stabilization, and in particular during emulsion treatment. Many practitioners support the idea that the added cement assists in the breaking of the emulsion. Another, and perhaps more viable, theory* is that the emulsion helps to bind the fines into a paste during the application of the emulsion, when the high moisture content involved would otherwise have lead to the segregation of the coarse and fine particles.

There is even some debate about the relative benefit of the bituminous binder, given the very low quantities of residual binder involved. Furthermore, depending on the cement content, traditional indicator tests such as UCS and ITS do not show a significant benefit from the addition of small bitumen contents (Long and Theyse, 2001).

However, it is crucial to note that laboratory studies cannot take into account the significant positive impact of bitumen stabilization during the construction process, where the right combination of bitumen and active filler provides fluid of the right consistency to optimize reworking and compaction of the material. It is perhaps possible that the main benefit of bitumen treatment is the transformation of the material, during construction, from a dry, non-pliable state to a viscous pliable state in which it can be readily reworked and compacted to a high density. If this material then works in a state that allows strength to develop under loading, a material with a high cohesion as well as a high friction resistance is obtained.

* This theory of the role of cement in bitumen treated materials was postulated by Mr Hugh Thompson of WSP Consulting Engineers.
It is clear that more fundamental research is needed to properly map the behaviour of bitumen stabilized materials, specifically when low binder contents are used, as is generally the case in southern Africa\(^\star\). At this stage, and in the absence of more detailed quantitative data, the following theory is put forward to describe the behaviour of bitumen stabilized materials, as commonly constructed in southern Africa, in the heuristic PN model:

- The behaviour of bitumen stabilized materials involving low binder contents is believed to be similar to that of an unbound granular material, but with a significantly improved cohesive strength. This behaviour places bitumen stabilized material somewhere between unbound granular material and intact cement stabilized material. The material behaviour will lean toward the latter when excessive cement is added.

- Bitumen stabilized materials perform well when cohesive strength is optimized through proper mix design (to determine the optimal binder and active filler contents), whilst retaining enough flexibility so that friction resistance is still activated under load.

- Similar to unbound granular materials, the stiffness of bitumen stabilized materials is dependent on the stiffness of the support. However, the high cohesive strength allows the material to sustain a higher modular ratio under loading (when compared to unbound granular materials).

- The maximum allowable stiffness of the material can vary significantly, depending on the relative amounts of bitumen and active filler added.

The above principles are assumed to hold for balanced mix designs with well graded source materials. Optimal shear strength of the material can be compromised in two ways:

- Excessive amounts of cement will transform the material from a somewhat flexible to a brittle state. In this state, the cohesive strength will dominate but will significantly reduce once fracture occurs. This is likely to be associated with deformation and cracking, and will result in a material consisting of large, fractured clumps, with a low frictional resistance.

- Poorly graded or non-durable source materials (e.g. soft weathered natural gravel or material with excessive fines) will compromise the frictional resistance of the material. Inexperienced designers may be tempted to compensate for such a situation through the addition of higher amounts of cement. Such fine grained, brittle material will be highly susceptible to crushing and fatigue failure.

\(^\star\) Such a study is currently being conducted by Prof. Kim Jenkins and co-workers at the University of Stellenbosch. The findings of the study are expected to be published during the pilot testing phase of the pavement design model described in this document.
6 CALIBRATION OF THE PN MODEL

In this section, the calibration of the relationship between the Pavement Number (PN) and pavement structural capacity is discussed. The calibration was based on three available data sets which, together with the rules of pavement behaviour outlined in Section 3, formed the knowledge base from which the PN based design model was calibrated. The three data sets comprise the following:

1. TRH4 Design Catalogue Set: this data set is comprised of structures extracted from the TRH4 design catalogue for Category A and B roads. The structures are those recommended for unbound base materials, to be used in wet and dry climates, and for traffic applications between 1 and 30 msa. This data set was used as a foundation on which to calibrate the climate adjustment factors, as well as the material constants for unbound and cement stabilized layers. This calibration was then refined based on the other two available data sets. More details on the use of this data set are provided in Section 6.1.

2. The LTPP Data Set for Bitumen Stabilized Pavements: this data set comprises all the identified in-service pavements that incorporate bitumen stabilized layers, and for which reliable historic pavement and traffic data could be obtained. Details of the development and use of this data set is provided in Section 6.2.

3. The HVS Data Set for Bitumen Stabilized Pavements: this data set comprises pavements that incorporate bitumen stabilized materials and which were tested using the Heavy Vehicle Simulator. Details of the development and use of this data set is provided in Section 6.3.

In the following subsections, the calibration of the PN design model using each of the three data sets is discussed. Basic aspects and assumptions related to each data set are first stated, followed by a discussion of the calibration process and key observations following from the calibration.

6.1 CALIBRATION USING THE TRH4 DESIGN CATALOGUE DATA SET

6.1.1 Building the Dataset

The TRH4 design catalogue (TRH4, 1996) together with the design principles documented in TRH4, has formed the mainstay of South African pavement design practice for approximately three decades. The catalogue is used for new designs and also to evaluate proposed rehabilitation designs. Although now quite outdated, it is believed that the TRH4 design catalogue still provides a valid – if somewhat conservative – foundation for the first level calibration of the PN model.

The structures selected from the TRH4 design catalogue include all those recommended for unbound base materials, to be used in wet and dry climates, and for traffic applications between 1 and 30 msa. For these situations, the catalogue recommends 17 pavement configurations. For each structure, the thickness of the selected layer should be determined based on the subgrade CBR. The subgrade situations and selected layer thicknesses, as prescribed in TRH4, are:

- Subgrade CBR 3 to 7 per cent (i.e. G10 material): 150 mm G7 plus 150 mm G9
- Subgrade CBR 7 to 15 per cent (i.e. G8 or G9 material): 150 mm
- Subgrade CBR above 15 per cent (i.e. G7 material): No selected layers needed

The above subgrade variations, when applied to the 17 selected pavement structures, provide a total of 51 pavement structures, each with an associated structural capacity. In the case of a G10 subgrade, a 300 mm G7 selected layer was assumed. This provides a slightly more conservative structure than that recommended by the TRH4.
In developing this data set, the assigned structural capacity was set to the upper limit of each pavement’s design class. Thus for a pavement recommended for a design class of 3 to 10 msa, the assigned capacity was 10 msa. This is an appropriate assumption, since all of the catalogue structures should be able to accommodate traffic ranging to the upper limit of the design class.

In some cases, the TRH4 catalogue provides an option on the surfacing type (either a seal or a thin asphalt could be used). In developing the data set, an asphalt surfacing of the recommended thickness was assumed in all instances. This was done to partly compensate for the higher tyre pressures being experienced on pavements in recent years, which makes the use of a surface seal somewhat risky in situations with high traffic intensity and/or lower quality base material.

For some structures, the catalogue also provides two options for the thickness of cement stabilized subbase layers depending on whether water is expected to enter the base or not. In these cases, the thickness adopted was the average of the lower and higher thicknesses recommended. The TRH4 Design Catalogue data set used for calibration and development of acceptance criteria is shown in Table 7.

6.1.2 Calibration

The TRH4 dataset was used for the first calibration of the PN model for unbound granular materials and cement stabilized subbases. The model was also used to calibrate the coefficient to use for adjusting the subgrade ELTS for wet climates. As such, this dataset is convenient to use for illustrating how the calibration was done. In the present context, the calibration process comprises the repeated adjustment of the model constants (as defined in Section 3), to yield an optimized correlation between PN and the allowed structural capacity.

In general, calibration proceeded as follows:

1. Best estimates for material constants were assumed based on the known behaviour and relative quality of each material type. The PN values for all structures were then calculated using the layer thicknesses and material classes shown in Table 5.

2. The PN values were plotted against the allowed or observed structural capacity (in general, pavement structures that belong to the same design category and design class tend to have similar PN values).

3. The PN versus Capacity plot (called a PN-Capacity plot) was examined and structures that did not meet the general pattern were identified and scrutinized to see which layers caused the deviation from the general pattern. The material constants were then adjusted and the process was repeated.

The process described in the above steps was run repeatedly, until an optimal set of constants were obtained. The adjustment of the constants was guided by judgement based on experience, and by the knowledge of how a particular constant would affect the PN.

As an example of the calibration process, Figure 18 shows the PN versus Capacity plot for a reasonable set of material constants, but with the subgrade factor for climate set to 1.0 (i.e. same factor as for dry regions). This means that effectively no adjustment of the subgrade stiffness was made to account for wet regions.
Table 7: Pavement Structures Used in the TRH4 Design Catalogue Dataset

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<th>Capacity (msa)</th>
<th>Design Category</th>
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<th>Subbase</th>
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<td>275 C4</td>
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<td>Dry</td>
<td>40</td>
<td>AC</td>
<td>125 G2</td>
<td>150 C4</td>
<td>150 G7</td>
<td>G4</td>
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<td>150 G7</td>
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<td>150 G7</td>
<td>G4</td>
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<td>Dry</td>
<td>40</td>
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<td>150 C4</td>
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<td>150 G7</td>
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<td>30</td>
<td>AC</td>
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<td>200 C3</td>
<td>150 G7</td>
<td>G4</td>
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<td>Wet</td>
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<td>AC</td>
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<td>30</td>
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<td>150 G7</td>
<td>G4</td>
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<td>Dry</td>
<td>40</td>
<td>AC</td>
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<td>250 C3</td>
<td>150 G7</td>
<td>G4</td>
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<td>Dry</td>
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<td>AC</td>
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<td>250 C3</td>
<td>150 G7</td>
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<td>47</td>
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<td>Dry</td>
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<td>AC</td>
<td>150 G1</td>
<td>250 C3</td>
<td>150 G7</td>
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<td>48</td>
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<td>Dry</td>
<td>50</td>
<td>AC</td>
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<td>150 G7</td>
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<td>49</td>
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<td>Wet</td>
<td>50</td>
<td>AC</td>
<td>150 G1</td>
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<td>AC</td>
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<td>350 C3</td>
<td>150 G7</td>
<td>G4</td>
</tr>
<tr>
<td>51</td>
<td>30</td>
<td>A</td>
<td>Wet</td>
<td>50</td>
<td>AC</td>
<td>150 G1</td>
<td>350 C3</td>
<td>150 G7</td>
<td>G4</td>
</tr>
</tbody>
</table>
Calibration of the PN Model

R² = 0.73

For illustration purposes, a trend line was fitted to this figure. Key observations that follow from Figure 18 are noted below:

- It can be seen that, even with the Category A and Category B roads combined in a single dataset, there is a good correlation between PN and the allowed capacity based on the TRH4 design catalogue. The coefficient of determination (R²) shows that the exponential relationship between capacity and PN explains more than 70 per cent of the variation in PN, even when there is no adjustment to take account of softer subgrades in wet areas.

- Despite the clear correlation between PN and capacity, it is clear that the pavements recommended for the wet regions are grouped separately, and consistently have greater PN values than dry region pavements in the same design class. This is to be expected, since we are not yet taking account of the (expected) weaker subgrade in the wet regions.

By decreasing the adjustment factor for the stiffness of the subgrade in wet regions, the correlation between PN and assigned capacity will improve. After repeated calculations using different climate adjustment factors, it was found that a climate adjustment factor of 0.6 for wet climates provides the best correlation between PN and capacity. Figure 19 shows the PN-Capacity plot for PN values determined with the improved climate adjustment factor. It is clear from this figure that the calibration of the subgrade adjustment factor leads to a significant improvement in the PN-Capacity relationship. This can be seen from the improved R² value and from the tighter grouping of data points for pavements recommended for wet and dry regions.

It should be noted that Figure 19 shows the data grouped only by climate, and not by road category. Figure 20 shows the pavements grouped by road category (combined for climate). This figure shows that, for the calibrated PN, there is a very strong correlation between PN and the structural capacity allowed by the TRH4 catalogue. It is clear that the calibrated PN is a good indicator of structural capacity, and can account for various subgrade situations, climatic regions, base and subbase types. For a given road category, there is a clear grouping of PN values for different traffic classes. This grouping of PN values for pavements with similar capacities provides a basis for the derivation of PN-based design criteria, which is discussed in detail in Section 7.
Calibration of the PN Model

\[ R^2 = 0.81 \]

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**Figure 19:** Illustration of Calibration Process (TRH4 dataset, subgrade adjusted for climate)

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**Figure 20:** PN versus Allowed Capacity for TRH4 Dataset, After Calibration
6.2 THE LTPP DATASET FOR BITUMEN STABILIZED PAVEMENTS

A key element of this study was the collection of performance information for pavements that incorporate bitumen stabilized layers. This task comprised the identification of roads in which bitumen stabilization was used, followed by the finding and compilation of all available information related to the construction, maintenance and performance of these pavements. This process yielded more than 25 roads for which the long term pavement performance (LTPP) could be assessed from available surveys and maintenance records.

A summary of the pavement configuration (i.e. layer thicknesses, layer types and material properties) and performance to date was compiled for each of the LTPP sections. This summary was then reviewed by practitioners who had been involved with aspects of the pavement design, construction and maintenance. The final reviewed summaries for each of the LTPP sections can be found in Long and Jooste (2007). These references provide detailed descriptions of the process of gathering, analyzing, synthesizing and verifying information related to the LTPP and HVS sections. A summary of the LTPP sections that have been summarized to date*, with key performance information, is shown in Table 8.

It should be noted that to compile the summary for each LTPP pavement situation, assumptions often had to be made owing to a lack of detailed information (especially related to the selected layers and subgrade). The following guidelines were followed when making these assumptions:

- For bituminous stabilized layers, a BSM1 material class was assigned when the source material was a crushed stone, a reworked, cement stabilized crushed stone or reclaimed asphalt pavement (RAP). Where natural gravel was used as a source material, a BSM2 class was assigned.

- For selected layers and subgrade, where detailed information on material properties was absent, the authors assumed values that seemed most realistic for the area, and given the performance of the pavement. In all cases the authors were sure to err on the conservative side♣.

- For cement stabilized layers that had been in service for some time at the time of the bitumen stabilization, it was generally assumed that the layer was still intact. This is again a conservative assumption. In some cases, where it was clear that the material had deteriorated to a granular state, a G6 material class was generally assigned. In some instances, where a thin cement stabilized layer appeared to be intact, but would clearly have been weakened owing to past traffic and construction activities, the material class was downgraded from a C3 to a C4.

- For each LTPP section, the past traffic was estimated based on historic records. Since key factors (such as the number of standard axles per heavy vehicle) were not known precisely, a range of factors were used in the analysis of past traffic♠. This provided an estimated range for the accumulated traffic that had been accommodated by the pavement. For the purposes of calibrating and validating the PN model, the lower range of the estimated past traffic was adopted. This is a conservative assumption, but it is regarded as prudent, considering that traffic loading intensity has increased significantly in recent years, and many of the LTPP sections carried relatively low traffic volumes in the years immediately after construction.

At the time of writing, five additional LTPP sections are in the process of being reviewed. These sections are: Road 1386 Moloto; TR16-3; N11-8; P243-1 and R22-4. Although the basic information for these sections has been gathered and compiled, summaries for these sections are not yet finalized, as the authors were awaiting feedback from reviewers. However, all five sections were less than 6 years old at the time of the review, and the findings from these sections will not impact on the calibration of the proposed method at this stage.

Since the pavement structures are used in calibrating the design model, a conservative assignment of a material class means that the better of two or more possible material classes was generally assumed where required.

Details of the historic traffic, including growth rates, number of heavy vehicles, etc. are provided with the section summaries. See Long and Jooste (2007) for details.
Table 8: Summary of Available LTPP Sections

<table>
<thead>
<tr>
<th>Project</th>
<th>Climate</th>
<th>Surface</th>
<th>Base</th>
<th>Subbase</th>
<th>Selected</th>
<th>Subgrade</th>
<th>Traffic (mesa)</th>
<th>Age (years)</th>
<th>Maintenance</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>N1 Section 1, Km 34.2 to 34.4 North and Km 34.8 to 35.0 South (Kraaifontein)</td>
<td>Mod</td>
<td>80 HMA</td>
<td>100 BSM1</td>
<td>100 C3</td>
<td>150 G5</td>
<td>G8</td>
<td>12 to 16</td>
<td>20</td>
<td>HMA surface replaced after 10 years</td>
<td>Sound</td>
</tr>
<tr>
<td>N7 Section 7, Km 47.0 to 47.2 North and South (Garies to Okiep)</td>
<td>Dry</td>
<td>Seal</td>
<td>150 BSM1</td>
<td>50 C3</td>
<td>150 G6</td>
<td>G8</td>
<td>1 to 1.5</td>
<td>18</td>
<td>Single seal after 8 years, Fog spray after 18 years</td>
<td>Sound</td>
</tr>
<tr>
<td>N1 Section 14, Km 6.0 to 19.0 (Springfontein to Trompsburg)</td>
<td>Dry</td>
<td>30 HMA plus Seal</td>
<td>150 BSM1</td>
<td>100 G4</td>
<td>300 G6</td>
<td>G7</td>
<td>10 to 13</td>
<td>25</td>
<td>Fog spray at 11 years, base patching and seal after 16 years</td>
<td>Warning</td>
</tr>
<tr>
<td>N1 Section 13, Km 45.4 to 48.6 (Springfontein to Trompsburg)</td>
<td>Dry</td>
<td>30 HMA plus Seal</td>
<td>150 BSM1</td>
<td>100 BSM2</td>
<td>300 G6</td>
<td>G7</td>
<td>10 to 13</td>
<td>25</td>
<td>Fog spray at 11 years, base patching and seal after 16 years</td>
<td>Warning</td>
</tr>
<tr>
<td>N3 Section 12, Km 36.3 to 40.8 South (Modderfontein to Buccleugh)</td>
<td>Mod</td>
<td>40 HMA</td>
<td>360 BSM1</td>
<td>150 C4</td>
<td>150 G7</td>
<td>G8 (?)</td>
<td>10 to 18</td>
<td>20</td>
<td>40 mm Overlay and surface repairs in some areas after 12 years</td>
<td>Sound</td>
</tr>
<tr>
<td>N3 Section 12, Km 34.5 to 36.0 North (Modderfontein to Buccleugh)</td>
<td>Mod</td>
<td>40 HMA</td>
<td>200 BSM1</td>
<td>200 C3</td>
<td>150 G9</td>
<td>G9</td>
<td>10 to 18</td>
<td>20</td>
<td>40 mm Overlay and surface repairs in some areas after 12 years</td>
<td>Sound</td>
</tr>
<tr>
<td>N2 Section 20, Km 31.7 to 37.1 (Tabankulu to Mintlawwa)</td>
<td>Wet</td>
<td>30 HMA</td>
<td>180 BSM1</td>
<td>270 G6</td>
<td>150 G7</td>
<td>G10</td>
<td>1 to 2</td>
<td>7</td>
<td>None to date</td>
<td>Sound</td>
</tr>
<tr>
<td>N2 Section 16, Km 38.5 to 38.7 and 37.6 to 37.8, North (Kwelerwa)</td>
<td>Wet</td>
<td>40 HMA</td>
<td>140 BSM2</td>
<td>130 C4</td>
<td>300 G7 (?)</td>
<td>G9 (?)</td>
<td>2 to 3</td>
<td>25</td>
<td>Seal placed at 24 years in service</td>
<td>Sound</td>
</tr>
<tr>
<td>N3 Section 4, Km 38.7 to 38.9 (Mooi River)</td>
<td>Wet</td>
<td>40 HMA</td>
<td>200 BSM1</td>
<td>150 C3</td>
<td>150 G7 (?)</td>
<td>G8</td>
<td>9 to 21</td>
<td>17</td>
<td>None reported</td>
<td>Warning</td>
</tr>
<tr>
<td>N4 Section 1, Km 19.3 to 25.6 (Scientia to Pienaars River)</td>
<td>Mod</td>
<td>30 HMA</td>
<td>170 BSM1</td>
<td>130 C4</td>
<td>300 G6</td>
<td>G7</td>
<td>3 to 4</td>
<td>9</td>
<td>None</td>
<td>Sound</td>
</tr>
<tr>
<td>N4 Section 5X, Km 20.0 to 25.0 (Wonderfontein to Crossroads)</td>
<td>Wet</td>
<td>30 HMA</td>
<td>150 BSM1</td>
<td>150 C3</td>
<td>150 G7 (?)</td>
<td>G9 (?)</td>
<td>2 to 5</td>
<td>9</td>
<td>None</td>
<td>Sound</td>
</tr>
<tr>
<td>N4 Section 5X, Km 27.6 to 29.0 (Wonderfontein to Crossroads)</td>
<td>Wet</td>
<td>30 HMA</td>
<td>150 BSM2</td>
<td>300 C3</td>
<td>150 G6</td>
<td>G9 (?)</td>
<td>1 to 5</td>
<td>9</td>
<td>None</td>
<td>Failed</td>
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</table>
### Calibration of the PN Model

<table>
<thead>
<tr>
<th>Project</th>
<th>Climate</th>
<th>Surface</th>
<th>Base</th>
<th>Subbase</th>
<th>Selected</th>
<th>Subgrade</th>
<th>Traffic (mesa)</th>
<th>Age (years)</th>
<th>Maintenance</th>
<th>Condition</th>
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<tbody>
<tr>
<td>N12 Section 19, Km 27.32 to Km 27.47 (Experimental Section 1)</td>
<td>Mod</td>
<td>55 HMA</td>
<td>120</td>
<td>120 G3</td>
<td>80 C3</td>
<td>G6</td>
<td>16 to 27</td>
<td>30</td>
<td>Fog spray after 17 years, crack sealing and patching started after 26 years in service</td>
<td>Warning</td>
</tr>
<tr>
<td>N12 Section 19, Km 27.17 to Km 27.32 (Experimental Section 2)</td>
<td>Mod</td>
<td>55 HMA</td>
<td>135</td>
<td>130 C3</td>
<td>220 G5</td>
<td>G6</td>
<td>16 to 27</td>
<td>30</td>
<td>Fog spray after 17 years, crack sealing and patching started after 26 years in service</td>
<td>Warning</td>
</tr>
<tr>
<td>P23-1, Km 3.0 to 4.0 (Kroonstad to Steynsrus)</td>
<td>Mod</td>
<td>35 HMA</td>
<td>150</td>
<td>150 G6</td>
<td>150 G8</td>
<td>G8 (?)</td>
<td>0.5 to 1.3</td>
<td>13</td>
<td>None</td>
<td>Failed</td>
</tr>
<tr>
<td>P24-1, Km 65.9 to 66.6 Westbound (Vereeniging)</td>
<td>Mod</td>
<td>30 HMA (B/R)</td>
<td>150</td>
<td>210 C4</td>
<td>300 G7 (?)</td>
<td>G9 (?)</td>
<td>2 to 5</td>
<td>7</td>
<td>None</td>
<td>Sound</td>
</tr>
<tr>
<td>Road 2388, Km 2.1 to 2.3 (Cullinan)</td>
<td>Mod</td>
<td>Slurry</td>
<td>100</td>
<td>150 G6</td>
<td>300 G6</td>
<td>G7 (?)</td>
<td>0.2 to 0.5</td>
<td>9</td>
<td>Double seals placed after 6 and 7 years in service</td>
<td>Warning / Failed</td>
</tr>
<tr>
<td>Same-Himo Road, Tanzania</td>
<td>Wet</td>
<td>Surface Seal</td>
<td>175</td>
<td>100 G6</td>
<td>150 G7 (?)</td>
<td>G8</td>
<td>1 to 2&lt;sup&gt;2&lt;/sup&gt;</td>
<td>8&lt;sup&gt;3&lt;/sup&gt;</td>
<td>None</td>
<td>Sound</td>
</tr>
<tr>
<td>MR27, Km 23.8 to 24.8, Slow Lane (Stellenbosch)</td>
<td>Mod</td>
<td>45 HMA plus seal</td>
<td>100</td>
<td>100 C3</td>
<td>150 G7 (?)</td>
<td>G8 (?)</td>
<td>4 to 9</td>
<td>20</td>
<td>Double seal with PMB after 14 years.</td>
<td>Good</td>
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<tr>
<td>MR504 Section A</td>
<td>Wet</td>
<td>Surface Seal</td>
<td>175</td>
<td>150 G6</td>
<td>150 G10</td>
<td>G10</td>
<td>0.7 to 2</td>
<td>12</td>
<td>Initially un-surfaced; 19 mm Seal placed after 3 years</td>
<td>Warning</td>
</tr>
<tr>
<td>MR504 Sections B and C</td>
<td>Wet</td>
<td>30 HMA</td>
<td>175</td>
<td>150 G6</td>
<td>150 G10</td>
<td>G10</td>
<td>0.7 to 2</td>
<td>12</td>
<td>Initially un-surfaced; 30 mm asphalt placed after 3 years</td>
<td>Warning</td>
</tr>
</tbody>
</table>

**Notes:**
1. A surface seal is assumed to have an effective thickness of 5 mm, used for purposes of adding effects of multiple seals and asphalt surfacings.
2. Sections that are highlighted are those in sound condition and that are believed to have carried traffic well below their design capacity. These sections were omitted from the data set in later analyses.
3. The range of accommodated traffic on the Same-Himo road after 8 years in service was reported to be between 1.4 (southbound lane) and 2.1 (northbound lane) mesa. The available data state that after 11 years in service the traffic accommodated was close to 4 million. The significant increase in loading over three years (i.e. between 8 and 11 years in service seems unlikely). It was therefore decided to adopt a more conservative estimate of accommodated traffic, especially in view of the fact that the road is outside southern Africa and the current condition could not be confirmed with certainty.
For each of the LTPP sections shown in Table 8, the PN was calculated and plotted against the number of equivalent standard axles accommodated. This process was again repeated using different material constants for BSM1 and BSM2 materials, until an optimal correlation between PN and capacity was obtained, whilst at the same time ensuring that the calibration constants were reasonable given the types of materials involved.

The resulting PN-Capacity plot is shown in Figure 21. This figure shows the data grouped in terms of the last recorded condition of the pavement. In interpreting this figure, it is important to note that the traffic accommodated by the sections in sound condition (green diamonds) represents the traffic accommodated to date. Since these pavements will continue to provide service for some time, the green symbols represent a snapshot of the current situation. However, effectively these green symbols are not fixed but are slowing moving upward as these pavements accommodate more traffic. It should also be noted that the points in Figure 21 represent the lower bound of the estimated range of traffic that the sections had accommodated.

![PN-Capacity Plot](image)

**Figure 21**: PN Values for LTPP Sections versus Traffic Accommodated

It can be seen from Figure 21 that there is a clear relationship between the traffic accommodated and the PN. This relationship is particularly clear for those pavements that are in warning condition and are therefore approaching the end of their structural lives. As expected, the sections still in sound condition plot mostly below the sections in warning condition. It is surprising to note that the three sections that are regarded as having failed, have accommodated the lowest traffic, and fall outside of the general trend. These three sections are marked A to C and are discussed in more detail below:

**Point A: Road D2388 near Cullinan**

Although this section is still in service, it has been indicated here as a failed section, owing to the poor riding quality and the apparent need for repeated resurfacing, despite the very low traffic accommodated by the section. A visual survey and rut depth measurements taken in 2005, one year after the second resal, showed a 90th percentile rut depth of approximately 8 mm with bleeding and undulation. This road was constructed using labour intensive construction techniques, and poor riding quality was apparent shortly after construction (Long and Jooste, 2007). It is thus clear that this road did not fail owing to normal structural deterioration, and thus the data point was removed from the set for purposes of further analyses.
Point B: Road P23-1 between Kroonstad and Steynsrus.

At the time when the section summary was compiled, the section had been in service for 13 years, and it is estimated that the section had accommodated between 0.5 and 1.3 mesas. The section was in a poor condition, with rut depths of between 10 and 20 mm in some parts. There were also patches which showed shoving in some areas. The relatively poor performance of this section is surprising, given the fact that this pavement is situated in a moderate climate, and has a structure comparable to other pavements in a wet climate that had accommodated more than 1 mesa*. The reason for the unexpectedly poor performance of this section is not clear. The structure is one of few structures in the knowledge base in which a bitumen stabilized layer (emulsion treated crushed dolerite in this case) was placed over an unbound gravel-sand support. It also represents an important design situation for recycling purposes. This data point was thus included in the dataset for further analyses.

Point C: N5 Section 5X, Km 27.6 to 29.0.

This section was constructed in 1996, and is in need of rehabilitation after 9 years in service owing to the presence of potholes and high rut depths. It is estimated that up to this time the road had accommodated between 1.3 and 5 mesas. The poor performance of this road is surprising, given the fact that the pavement structure was substantial, with two 150 mm C4 subbase layers according to the as-built records. By comparison, N4-5X, Km 20 to 25, was constructed using a single 150 mm C3 subbase, and is still in a good condition after having accommodated between 2 and 5 mesas.

However, one striking factor for this road is that it is one of the few cases in which an emulsion treated natural gravel was used for a design that involved fairly intense traffic loading. This raises some concern about the ability of a treated natural gravel to be used in design situations that involve heavy traffic (typically road with a design traffic of 10 mesas or greater). However, it should also be noted that informal discussions with persons that had been involved with the project during construction suggested that the construction had not been of a high standard, with the existing pavement structure and source material varying significantly.

Overall, given the available information and the substantial pavement structure involved, it is believed that this section represents a premature failure that is not related to the pavement structure. However, this data point was thus included in the data set for further analyses, and questions raised by this section will be discussed further in Section 8.

Several of the pavements shown Figure 21 are in sound condition and have accommodated very little traffic (These sections are highlighted in Table 8). Based on the general PN-structural capacity relationship, it is believed that these pavements have accommodated much less traffic than allowed by the pavement structural capacity and therefore do not facilitate an assessment of the likely design capacity. To clarify the data set for further analyses, these sections were thus omitted from the dataset. Figure 22 shows the PN-Capacity relationship for this reduced dataset.

* Compare, for example, the P23-1 structure with the structure of N2 Section 20, MR504 and the Same-Himo project, all of which have accommodated roughly 1 mesa, and are in a better condition in general.
6.3 THE HVS DATASET FOR BITUMEN STABILIZED PAVEMENTS

In addition to the LTPP sections described in the previous subsection, seven bitumen stabilized pavements in South Africa have been subjected to testing with the Heavy Vehicle Simulator. The HVS test sections yielded 15 pavement configurations for which relatively detailed information on the pavement structure and accommodated HVS loading could be obtained. In addition, the performance of the pavement under HVS testing was evaluated and summarized. These summaries can also be found in Long and Jooste (2006). Key aspects of each of the HVS test sections are shown in Table 9.

It should be noted that – as with the LTPP sections – certain assumptions had to be made regarding the material types and performance of the HVS sections. To determine the material types involved in each HVS test section, the same guidelines as stated earlier for LTPP sections were followed.

To determine the number of equivalent standard axles accommodated, the applied axles were converted by using damage exponents of 3.5, 4.0 and 4.5. Using these three damage exponents, the applied axle loading, which ranged from 80 kN to greater than 200 kN, could be converted to equivalent standard axles. To summarize section performance, the middle of the range (i.e. using a damage exponent of 4.0), was adopted.

This assumption is different to the one applied to the LTPP data set, where the lower limit of the estimated traffic range was assumed. The reason for this is the very aggressive form of loading that was applied in most HVS tests, which included axle loading of 200 kN or more in many instances. In view of this aspect, coupled with the observation that many HVS sections were in a sound condition at the stage when water was added to the section, it was felt that the use of the lower bound traffic estimate would be over-conservative in the case of HVS trafficking. The middle of the range of estimated traffic was therefore adopted to represent traffic on HVS sections.
Table 9: Summary of Available HVS Sections

<table>
<thead>
<tr>
<th>Project</th>
<th>Test State</th>
<th>Surface</th>
<th>Base</th>
<th>Subbase</th>
<th>Selected</th>
<th>Subgrade</th>
<th>Traffic (mesa)</th>
<th>Performance</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>N3 Section 4 (Mooi River) Test Section 98A3</td>
<td>Dry</td>
<td>90 HMA</td>
<td>200 BSM1</td>
<td>300 C4</td>
<td>150 G7 (?)</td>
<td>G8 (?)</td>
<td>10 – 15 – 23</td>
<td>15 mm Rut. Controlled temperature of 30 to 35°C</td>
<td>Warning</td>
</tr>
<tr>
<td>N3 Section 4 (Mooi River) Test Section 99A3</td>
<td>Dry</td>
<td>90 HMA</td>
<td>200 BSM1</td>
<td>150 G5</td>
<td>150 G7 (?)</td>
<td>G8 (?)</td>
<td>8 – 12 – 19</td>
<td>23 mm Rut. Controlled temperature of ± 35°C</td>
<td>Failed</td>
</tr>
<tr>
<td>N3 Section 4 (Mooi River) Test Section 100A3</td>
<td>Wet</td>
<td>90 HMA</td>
<td>200 BSM1</td>
<td>150 C4</td>
<td>150 G7 (?)</td>
<td>G8 (?)</td>
<td>4 – 6 – 8</td>
<td>33 mm Rut. Water entered through cracks during heavy rain</td>
<td>Failed</td>
</tr>
<tr>
<td>N2 Section 16 (Kwelerwa, East London) Test Section 322A2</td>
<td>Dry</td>
<td>60 HMA</td>
<td>100 BSM2</td>
<td>130 C4</td>
<td>100 G7 (?)</td>
<td>G9 (?)</td>
<td>5 – 8 – 12</td>
<td>15 mm Rut</td>
<td>Warning</td>
</tr>
<tr>
<td>P9-3 (Heilbron)</td>
<td>Dry</td>
<td>30 HMA</td>
<td>150 BSM2</td>
<td>150 C4</td>
<td>300 G7</td>
<td>G9 (?)</td>
<td>5 – 10 – 15</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>D2388 (Cullinan) Test Section 397A4</td>
<td>Dry</td>
<td>30 mm HMA</td>
<td>100 BSM2</td>
<td>300 G6</td>
<td>300 G7</td>
<td>G9 (?)</td>
<td>1 – 2 – 3</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>D2388 (Cullinan) Test Section 403A4</td>
<td>Dry</td>
<td>30 mm HMA</td>
<td>150 BSM2</td>
<td>300 G6</td>
<td>300 G7</td>
<td>G9 (?)</td>
<td>1 – 2 – 3</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>D2388 (Cullinan) Test Section 407A4</td>
<td>Dry</td>
<td>30 mm HMA</td>
<td>150 BSM2</td>
<td>300 G6</td>
<td>300 G7</td>
<td>G9 (?)</td>
<td>11 – 15 – 22</td>
<td>20 mm Rut in dry condition</td>
<td>Failed</td>
</tr>
<tr>
<td>P243-1 (Vereeniging), Test Sections 409A4</td>
<td>Dry</td>
<td>25 HMA</td>
<td>275 BSM2</td>
<td>60 G6</td>
<td>150 G7 (?)</td>
<td>G9 (?)</td>
<td>7 – 11 – 17</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>P243-1 (Vereeniging), Test Sections 410A4</td>
<td>Dry</td>
<td>25 HMA</td>
<td>275 BSM2</td>
<td>60 G6</td>
<td>150 G7 (?)</td>
<td>G9 (?)</td>
<td>8 – 11 – 17</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>P243-1 (Vereeniging), Test Sections 411A4</td>
<td>Dry</td>
<td>25 HMA</td>
<td>275 BSM2</td>
<td>60 G6</td>
<td>150 G7 (?)</td>
<td>G9 (?)</td>
<td>5 – 6 – 9</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>P243-1 (Vereeniging), Test Sections 412A4</td>
<td>Dry</td>
<td>25 HMA</td>
<td>275 BSM2</td>
<td>60 G6</td>
<td>150 G7 (?)</td>
<td>G9 (?)</td>
<td>5 – 7 – 9</td>
<td>Less than 5 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>N7 (TR11/1), Test Section 415A5</td>
<td>Dry</td>
<td>55 HMA</td>
<td>250 BSM1</td>
<td>120 G3</td>
<td>150 G8</td>
<td>150 G8</td>
<td>6 – 9 – 13</td>
<td>8 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>N7 (TR11/1), Test Section 416A5</td>
<td>Dry</td>
<td>55 HMA</td>
<td>250 BSM1</td>
<td>120 G3</td>
<td>150 G8</td>
<td>150 G8</td>
<td>6 – 8 – 11</td>
<td>6 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
<tr>
<td>N12-19 (Daveyton), Test Section 425A5</td>
<td>Dry</td>
<td>55 HMA</td>
<td>135 BSM1</td>
<td>160 C3</td>
<td>200 G5</td>
<td>G6</td>
<td>31 – 42 – 59'</td>
<td>Less than 10 mm Rut in dry condition</td>
<td>Sound</td>
</tr>
</tbody>
</table>

Refer to the text for an explanation of numbered notes. Highlighted sections are believed to represent sections that had not been tested to the limits of their capacity, and were excluded from some of the analyses.
The following annotations apply to Table 8:

1. Since HVS tests are performed in an accelerated mode of loading, the influence of natural moisture ingress is negligible if the surface is impermeable and water is not artificially added. Thus in all cases a dry test situation was assumed, except in cases where there was a clear sign of water ingress owing to a cracked surface and high rainfall during the test.

2. Cases where little or no information was available on the thickness and/or material type for a specific layer are marked as “(?).” In these cases, a conservative estimate of material quality (i.e. a better material class) was generally assumed.

3. The range of traffic shown represent the rounded equivalent standard axles, in millions (mesa), calculated from the applied axle loading, and using damage exponents of 3.5, 4.0 and 4.5. Thus a value 2-6-12 indicates that the traffic loading was 2, 6 and 12 mesa for damage exponents of 3.5, 4.0 and 4.5, respectively.

4. Unless otherwise noted, the performance summary applies to the pavement in a dry state, before water was added to or forced into the section.

5. In the case of the P9-1 (Heilbron) HVS section, different percentages of emulsion were used (Long and Jooste, 2006). However, the performance of all sections was similar, except for one section which had a very low binder content (0.6% residual binder). The latter section is not considered a normal treatment situation, and was thus excluded from the interpretation of this HVS test. The other sections accommodated various axle loading situations. In all instances the rut depth that developed during testing in the dry state was below 5 mm. The traffic shown therefore represents a conservative summary of the traffic that had been accommodated.

6. In the case of the Cullinan HVS test (road D2388), the subbase was specified to be a cement stabilized layer. However, test pits showed that this layer was not properly stabilized and in some cases a DCP penetration rate of up to 5 mm per blow was measured within this layer (Long and Jooste, 2006). For purposes of this assessment, the layer was regarded as a unbound sand-gravel blend. One test section (408A4) was subjected to very little traffic in the dry state, with less than 5 mm rut. This section is omitted for purposes of this interpretation, since the capacity limit was clearly not reached.

7. For the HVS test on N12 Section 19, the estimated normal road traffic that had accumulated before the HVS test was added to the HVS loading. This traffic represents roughly 11.8 mesa (Jones, 2004). The subgrade on this section was an exceptionally dense and stiff material, and appropriate material classes were accordingly assumed for this section.

Figure 23 shows the calculated PN values plotted against the middle of the range of estimated traffic on the HVS sections. In this case, the relationship between the calibrated PN and the axles accommodated is weaker compared to the LTPP sections (Figure 22). This is to mostly due to the relatively poor performance of the three HVS sections tested on N3-4 (a relatively stiff and deep structure) and to the relatively good performance of P243-1 (a relatively light structure). It should also be noted that some HVS test sections (e.g. two of the D2388 Cullinan test sections), had clearly not been tested to the limits of their capacity.

According to the PN value, the N4-3 HVS sections should show a relatively high capacity under loading. However, for these sections, much (if not all) of the rutting originated in the 90 mm asphalt surfacing which was subjected to a constant, high temperature during testing. Investigators involved in the testing of these sections also noted that some experienced practitioners were not satisfied with the construction of these sections, and felt the sections do not represent typical bitumen stabilized sections (Long and Jooste, 2006).
Figure 23: Axles Accommodated versus PN for HVS Test Sections

6.4 SUMMARY

In this section, the process of calibrating the PN to provide an optimal relationship with structural capacity was discussed. Three data sets were used in the calibration process: (a) a data set built using TRH4 design catalogue structures; (b) a data set comprising LTPP sections; and (c) a data set comprising sections tested with the HVS.

It was shown that the calibrated PN is well correlated with the structural capacity prescribed by the TRH4 design catalogue, and also shows a reasonable correlation with the traffic that had been accommodated by LTPP and HVS sections. The correlation between structural capacity and PN is weaker for the HVS data set than for the TRH4 and LTPP data sets. This aspect will be discussed in more detail in the following section, which discussed the development of design criteria for the PN on the basis of the three available data sets.
7 DEVELOPMENT OF DESIGN CRITERIA

In the previous section, the three data sets used for calibration of the PN-related constants were described. It was also shown that the PN exhibits a reasonable correlation with the observed or prescribed structural capacity of pavements in the three data sets. In this section the, development of design criteria is discussed, again using the three available data sets described in Section 6.

7.1 REFINEMENT OF THE DATA SET

Figure 24 shows the calculated PN value for all available data, plotted versus the allowed (in the case of TRH4 pavements) or observed (in the case of LTPP or HVS sections) capacity. In this figure, the PN value was adjusted by a random number between 0 and 1, to ensure that points with the same PN number and capacity do not plot over one another. The data shown in Figure 24 includes all available points, including those sections which were relatively new, or had failed prematurely.

Figure 24 shows a clear relationship between PN and structural capacity. The strength of the PN-Capacity relationship is surprising, given that many of the pavements (represented by green symbols) had not yet reached the end of their functional design life.

![Figure 24: PN versus Capacity for Full Combined Data Set](image-url)
Figure 25 shows the PN-Capacity relationship for a reduced data set, in which the sections which were relatively new, or which had failed prematurely owing to reasons not related to the structural capacity, were removed. The sections that were removed were discussed in Section 4, but are briefly restated here for completeness:

- Sections N11-8, N2-20, N4-1, P24-1, P243/1, R22-4, TR 16-3 and Road 1386 have been in service for between 3 and 9 years and had accommodated little traffic to date. Section N7-7 has been in service for more than 10 years, but the section is in sound condition and had carried relatively little traffic.

- Two of the HVS test sections on road D2388 near Cullinan were removed as these sections were not tested to the limits of their capacity whilst in the dry state.

- The LTPP section representing road D2388 near Cullinan was removed since the failure of this section is almost certainly due to poor construction and surface related distress.

- The HVS tests on Section N4-3 were removed since these sections were reportedly not well constructed. In addition, HVS testing was done at high temperatures and much of the rutting had originated in the thick asphalt surfacing.

Figure 25 shows a clear relationship between PN and capacity, and provides the basis for the derivation for acceptance criteria for the PN, which is discussed in the following subsection.

![Figure 25: PN versus Capacity for the Reduced Data Set](image-url)
7.2 Derivation of Structural Capacity Criteria

After a detailed analysis of the trends exhibited by the data shown in Figure 25, structural capacity criteria in the form of a step function were derived. At this stage, the criteria were developed only for Category A and B roads, and for pavements with design capacities between 1 and 30 mesa. This restriction stems from the lack of data for pavements that have accommodated traffic outside these limits. For most design situations, a Category A or B assessment would be sufficient for roads with traffic above 1 mesa. However, it is also proposed that a design catalogue be developed at the conclusion of the pilot testing phase of this project, and this catalogue should provide guidance for Category C and D design situations.

As with the calibration process, the derivation of these criteria relied on an iterative process. In this case the objective was to obtain criteria which would lead to an optimal assessment of the structural capacity of each of the pavements represented in Figure 17. A step function was chosen since this function is robust and not sensitive to small changes in PN (and thus also layer input values). The step function could also be more closely fitted to the data than an exponential or other complex mathematical function. The step function criteria are evaluated as follows:

\[ N_{\text{allow}} = N_1 + (PN - PN_1) \times \text{Slope} \]

(Equation 5.1)

Where:
- \( N_{\text{allow}} \) = Allowed pavement capacity
- \( N_1 \) = Lower limit for the capacity range from Table 10 or Table 11
- \( PN \) = Calculated pavement number
- \( PN_1 \) = Lower limit for the PN range from Table 10 or Table 11
- \( \text{Slope} \) = Slope for the PN range from Table 10 or Table 11

The values \( N_1 \), \( PN_1 \) and \( \text{Slope} \) are obtained from Table 10 or Table 11 (depending on the road category), after first determining the range within which the calculated PN falls. Figure 26 shows the acceptance criteria imposed on the reduced data set.

**Table 10: PN Criteria for Category A Roads**

<table>
<thead>
<tr>
<th>PN Range</th>
<th>( N_1 )</th>
<th>PN1</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>PN &lt; 15</td>
<td>Less than 3 mesa, not suited for Category A roads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15 &lt; PN ≤ 23</td>
<td>3</td>
<td>15</td>
<td>0.00</td>
</tr>
<tr>
<td>23 &lt; PN ≤ 25</td>
<td>3</td>
<td>23</td>
<td>3.50</td>
</tr>
<tr>
<td>25 &lt; PN ≤ 32</td>
<td>10</td>
<td>25</td>
<td>0.00</td>
</tr>
<tr>
<td>32 &lt; PN ≤ 35</td>
<td>10</td>
<td>32</td>
<td>6.67</td>
</tr>
<tr>
<td>PN &gt; 35</td>
<td>30</td>
<td>35</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Table 11: PN Criteria for Category B Roads**

<table>
<thead>
<tr>
<th>PN Range</th>
<th>( N_1 )</th>
<th>PN1</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>PN &lt; 3</td>
<td>Less than 1 mesa, use Design Catalogue</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 &lt; PN ≤ 8</td>
<td>1</td>
<td>3</td>
<td>0.00</td>
</tr>
<tr>
<td>8 &lt; PN ≤ 11</td>
<td>1</td>
<td>8</td>
<td>0.67</td>
</tr>
<tr>
<td>11 &lt; PN ≤ 15</td>
<td>3</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>15 &lt; PN ≤ 25</td>
<td>3</td>
<td>15</td>
<td>0.70</td>
</tr>
<tr>
<td>PN &gt; 25</td>
<td>Use Category A Criteria</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
7.3 Validation of the PN-Model and Proposed Criteria

To assess the accuracy and the suitability of the proposed criteria, the allowed structural capacity was evaluated for each of the pavements in the reduced LTPP and HVS data sets. The outcomes of these evaluations are summarized in Table 12 and Table 13 for pavements with unbound and bound subbases, respectively.

Table 12 and Table 13 show that the PN-model and proposed criteria provides an appropriate classification of the structural capacity of almost all of the pavements in the LTPP and HVS data sets. In some cases, the PN-assigned structural capacity seems overly conservative, especially for HVS test on N12 Section 19, which had accommodated significant amounts of traffic. It should be taken into account, however, that, in the development of the PN criteria, care was taken to base the criteria on general trends, and not on one or two outstanding cases.

The HVS test on N12 Section 19 is regarded as a special case, since the road is had “aged gracefully” under slowly increasing traffic up to the point when the HVS test was started. The LTPP sections on this road perhaps provide a more realistic indication of the road’s capacity, and for both the LTPP sections the PN-based capacity assignment seems appropriate.

Overall, it is believed that the evaluation of model outcomes for the LTPP and HVS test sections validate the accuracy and suitability of the PN model and proposed criteria. Overall, the assessment of the proposed criteria shows that the criteria, when used in conjunction with the calculated pavement number, are relatively accurate and suitably conservative.

Apart from two roads, the proposed criteria in all cases are conservative for pavements in a warning or failed condition. At this stage, there is no clear explanation for the two roads which appear to have failed before the allowed structural capacity was accommodated. However, these two roads clearly represent a deviation from the general PN-Capacity relationship, and at this stage it is recommended that the proposed criteria be accepted for purposes of pilot implementation.
Table 12: Validation of PN Model and Proposed Criteria for Pavements with Unbound Subbases

<table>
<thead>
<tr>
<th>Source</th>
<th>Project</th>
<th>Climate or Test State</th>
<th>Traffic Accommodated* (mesa) and Age</th>
<th>Condition</th>
<th>PN</th>
<th>Allowed Capacity (msa)</th>
<th>Comments on Capacity Assignment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Cat A</td>
<td>Cat B</td>
</tr>
<tr>
<td>LTPP</td>
<td>N1 Section 14 (Springfontein to Trompsburg)</td>
<td>Dry</td>
<td>10 to 13 (25 years)</td>
<td>Warning</td>
<td>25</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>N12 Section 19, Experimental Section 1</td>
<td>Moderate</td>
<td>16 to 27 (30 years)</td>
<td>Warning</td>
<td>33</td>
<td>17</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Same-Himo Road, Tanzania</td>
<td>Wet</td>
<td>1 to 2 (8 years)</td>
<td>Sound</td>
<td>7</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>MR504 Section A</td>
<td>Wet</td>
<td>0.7 to 2 (12 years)</td>
<td>Warning</td>
<td>5</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>MR504 Section B</td>
<td>Wet</td>
<td>0.7 to 2 (12 years)</td>
<td>Warning</td>
<td>5</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>P23-1 (Kroonstad to Steynsrus)</td>
<td>Moderate</td>
<td>0.5 to 1.3 (13 years)</td>
<td>Failed</td>
<td>14</td>
<td>N/A</td>
<td>3</td>
</tr>
<tr>
<td>HVS Test</td>
<td>D2388 Cullinan Test 397A4</td>
<td>Dry</td>
<td>1 to 3</td>
<td>&lt; 5 mm Rut</td>
<td>18</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>D2388 Cullinan Test 403A4</td>
<td>Dry</td>
<td>1 to 3</td>
<td>&lt; 5 mm Rut</td>
<td>19</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>D2388 Cullinan Test 407A4</td>
<td>Dry</td>
<td>11 to 22</td>
<td>20 mm Rut</td>
<td>19</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>P243 Vereeniging (all test sections)</td>
<td>Dry</td>
<td>5 to 10+</td>
<td>&lt; 5 mm Rut</td>
<td>15</td>
<td>N/A</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>N7-7 (TR11/1), all test sections</td>
<td>Dry</td>
<td>6 to 10+</td>
<td>&lt; 10 mm Rut</td>
<td>31</td>
<td>N/A</td>
<td>10</td>
</tr>
</tbody>
</table>

* Traffic accommodated to date and not traffic to failure.
<table>
<thead>
<tr>
<th>Source</th>
<th>Project</th>
<th>Climate or Test State</th>
<th>Traffic Accommodated* (mesa) and Age</th>
<th>Condition</th>
<th>PN</th>
<th>Allowed Capacity (msa)</th>
<th>Comments on Capacity Assignment</th>
</tr>
</thead>
<tbody>
<tr>
<td>LTPP</td>
<td>N1 Section 1 (Kraaifontein)</td>
<td>Moderate</td>
<td>12 to 16 (20 years)</td>
<td>Sound</td>
<td>29</td>
<td>10</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td></td>
<td>N1 Section 13 (Springfontein to Trompsburg)</td>
<td>Dry</td>
<td>10 to 13 (25 years)</td>
<td>Warning</td>
<td>26</td>
<td>10</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td></td>
<td>N3 Section 12, 36.3 to 40.8 South (Modderfontein to Bucleugh)</td>
<td>Moderate</td>
<td>10 to 18 (20 years)</td>
<td>Sound</td>
<td>35</td>
<td>30</td>
<td>Seems appropriate. More monitoring is needed to confirm.</td>
</tr>
<tr>
<td></td>
<td>N3 Section 12, 34.5 to 36.0 North (Modderfontein to Bucleugh)</td>
<td>Moderate</td>
<td>10 to 18 (20 years)</td>
<td>Sound</td>
<td>26</td>
<td>10</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td></td>
<td>N2 Section 16 (Kwelera)</td>
<td>Wet</td>
<td>2 to 3 (25 years)</td>
<td>Sound</td>
<td>17</td>
<td>3</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td></td>
<td>N3 Section 4 (Mooi River)</td>
<td>Wet</td>
<td>9 to 21 (17 years)</td>
<td>Warning</td>
<td>24</td>
<td>7</td>
<td>Good. Suitably conservative</td>
</tr>
<tr>
<td></td>
<td>N4 Section 5X, Km 26.5 to 29.0 (Wonderfontein to Crossroads)</td>
<td>Wet</td>
<td>1 to 5 (9 years)</td>
<td>Failed</td>
<td>24</td>
<td>7</td>
<td>Assessment seems appropriate, but obviously did not identify risk for premature failure.</td>
</tr>
<tr>
<td></td>
<td>N12 Section 19, Experimental Section 2</td>
<td>Mod</td>
<td>16 to 27 (30 years)</td>
<td>Warning</td>
<td>29</td>
<td>10</td>
<td>Fair, perhaps over-conservative.</td>
</tr>
<tr>
<td></td>
<td>MR27 Stellenbosch</td>
<td>Mod</td>
<td>4 to 9 (20 years)</td>
<td>Sound</td>
<td>20</td>
<td>3</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td>HVS Test</td>
<td>N2 Section 16 (Kwelera)</td>
<td>Dry</td>
<td>5 to 12</td>
<td>15 mm Rut</td>
<td>19</td>
<td>3</td>
<td>Good. Suitably conservative.</td>
</tr>
<tr>
<td></td>
<td>P9-3 Heilbron</td>
<td>Dry</td>
<td>5 to 15</td>
<td>&lt; 5 mm Rut</td>
<td>17</td>
<td>3</td>
<td>Fair, perhaps over-conservative.</td>
</tr>
<tr>
<td></td>
<td>N12-19 Daveyton</td>
<td>Dry</td>
<td>31 to 59</td>
<td>&lt; 10 mm Rut</td>
<td>30</td>
<td>10</td>
<td>Seems over-conservative when considering traffic, but not when the structure as a whole is considered.</td>
</tr>
</tbody>
</table>

* Traffic accommodated to date and not traffic to failure.
8 PRACTICAL AND PERIPHERAL DESIGN CONSIDERATIONS

The preceding sections presented a methodology to determine the structural capacity of pavements incorporating bitumen stabilized materials. The method uses the Pavement Number to quantify the overall long term load spreading capacity of the pavement. It was shown that the PN is well-correlated to expected structural capacity for structures in the TRH4 design catalogue.

The method is also well correlated to the observed structural capacity of pavements that have been in service for longer than 10 years. This is evidenced by the trends shown in Figure 22 (repeated here as Figure 27 for convenience), which shows a high coefficient of determination ($R^2$), for pavements in a warning condition. This suggests that the calibrated PN model explains more than 90 per cent of the variation in observed structural capacity for these pavements.

Design criteria for the PN-model were also derived and it was shown that the model is well-validated for almost all pavements in the LTPP and HVS data sets. Of concern, however, are the three pavements that are considered to have failed, and which are shown as red dots in Figure 27. What is immediately striking about these three pavements is the fact that they all accommodated relatively low traffic before failure, regardless of the PN value.

One explanation may be that the PN does not adequately capture the structural capacity of certain pavements. However, since the relationship between PN and structural capacity is clear for most of the other pavements in the overall dataset, it is highly unlikely that by coincidence the PN is inappropriate only for the three pavements that have failed.

The clear deviation of the failed pavements from the general trend shown in Figure 27 suggests that another failure mechanism is at work for these three pavements. It is the authors’ opinion that the three failed pavements shown in Figure 27 represent premature failures, and that these failures involve a different deterioration mechanism compared to the other structures shown in this figure.

In the following subsections, the concept of premature failure will be explored in more detail. Basic concepts of risk and reliability will be discussed, whereafter the main risk elements will be discussed. A risk management system for prevention of premature failures is proposed, with recommendations on how to refine such a system for incorporation in the finalized design guidelines.

![Figure 27: PN versus Traffic Accommodated (reduced LTPP data set)](image_url)
8.1 **Premature Failure versus Wearout Failure**

One of the concepts of reliability theory is that the reliability of a system often depends on the time it has been in service (Miller et al., 1990)*. Conceptually, this situation can be depicted by the failure rate curve illustrated in Figure 28. The failure rate curve is typically divided into three parts which represent the following three stages of a product or system’s service life:

1. In the first stage of service life, there is initially a higher risk of failure, but this decreases over time. The higher rate of failure is caused by poorly manufactured products.

2. The second stage sees a constant failure rate, and represents the bulk of a product's useful service life.

3. The third part of the failure rate curve represents the stage when products fail primarily because they are worn out. In essence, the onset of this stage represents the end of a product's design life.

![Figure 28: A Typical Failure-Rate Curve (after Miller et al. 1990)](image)

In the context of pavement design, the structural design life of a pavement determines roughly when the third stage is likely to occur. Naturally, the design method should ensure that the pavement structure is adequate so that wearout will not occur within the design period. In essence, a structural design method aims to determine where point A, as shown in Figure 28, is situated.

8.2 **Accumulated Risk**

Pavement design methods are generally concerned only with determining the onset of wearout failures, and are not concerned with early failures or the risk of these failures occurring. Pavement designers normally regard early failure as a contractor problem, and often disputes over premature pavement failure will centre around the question as to whether the failure was caused by poor construction (the designer's viewpoint) or by poor design which caused point A (in Figure 28) to move to a point much earlier in the service life period (typically the contractor’s viewpoint).

* The discussion of the basic concepts in this subsection is closely based on that given in Miller et al. (1990).
Whilst poor design and poor construction will always remain, one factor that is often overlooked is the combined risk induced by marginal decisions or risk factors which enter at various stages of the overall production process. In the context of pavement design and construction, this process can be summarized by the several main steps, as illustrated in Figure 29.

**Pavement Investigation**, including layer thickness determination and materials classification

**Pavement Design**, including selection of road category or design confidence

**Determination of Design Traffic**

**Materials Design**, including binder content determination and specifications

**Construction**, including quality control, trial sections and supervision aspects

**Figure 29: Steps in Design and Construction Process**

Consider now the following scenario, assumed to apply to a cold-mix recycling design situation over a 1.5 km length:

1. Owing to lack of funds, only two trial pits are opened in the design section. A 470 m section which consists of a different type of base material, consisting of less durable aggregate and more plastic fines, is not identified.

2. The designer performs a suitable pavement design, but after discussion with the client, decides to design for the middle of the expected range of design traffic.

3. There is very little time to do a proper mix design, and the design relies mainly on indicator tests. The designer is experienced with pavement design, but has only performed one mix design involving bitumen stabilized materials.

4. The contractor is experienced but under financial pressure. Owing to lack of funding, there is no permanent resident engineer (RE) on site, but an experienced RE working in the vicinity is asked to visit the site at least three times per week.

In the above scenario, several risk factors enter into the design-and-construction process. None of these factors, in isolation, will necessarily result in premature failure. In fact, a suitably conservative pavement and mix design should effectively nullify the risk of premature failure from any one of these factors. However, when these factors are considered in combination, the risk of failure is increased significantly, and the above scenario is almost certain to be a recipe for a premature failure.

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This lack of taking into account risk across all elements of the design-and-construction process can be addressed, and a knowledge based system is especially suited to achieve this. In essence, this means that the structural design and specification processes should be closely linked, and that designers should be intimately aware of risk factors during the structural design process. The following subsections highlight and discuss key aspects that are deemed to be risk factors in the design and construction of pavements with bitumen stabilized layers.
8.3 SURFACING TYPE

One aspect that emerged clearly from the analysis of the LTPP dataset was that very few roads were constructed with a surface seal. In general, and even for roads with design traffic as low as 1 mesa, a thin to medium asphalt surfacing, often combined with a single seal, was used. The general trend of surfacing thickness versus allowed or observed capacity is shown in Figure 30.

This figure shows pavements from all three data sets in the knowledge base (TRH4, LTPP and HVS). The figure clearly shows that, for traffic up to roughly 10 mesa, the surfacing consists mainly of hot mix asphalt with a thickness that varies between 25 and 40 mm*. For pavements that have accommodated (or should be able to accommodate, in the case of the TRH4 data set) more than 10 mesa the combined surfacing thickness is almost always above 50 mm.

Based on the observations in the pavement knowledge base, it is therefore strongly recommended that bituminous stabilized layers always be combined with an asphalt surfacing. An exception can perhaps be made in the case of pavements with design traffic of less than 1 mesa. The dotted line in Figure 30 shows a recommended minimum surfacing thickness. This thickness can be reduced by 5 mm if a surface seal is placed in combination with the asphalt surfacing.

It should be kept in mind that recycling with bitumen stabilization almost always involves a reworking of the original base material. Although fine material may be added to the recycled material to improve the grading, the main load bearing component should remain the coarse aggregate, the strength of which is unlikely to be as high as that of a newly crushed material. Crushing of the recycled aggregate, resulting in deformation and formation of a weak lens beneath the surfacing, is thus a real possibility in pavements involving recycled layers, and it is believed that a thin to medium thick asphalt surfacing can greatly alleviate this risk.

* In Figure 30, surface seals are indicated with a 5 mm thickness.
8.4 IMPORTANCE OF SOURCE MATERIAL

Another observation that followed from the analysis of the LTPP dataset is the tendency to use higher quality source materials for the bitumen stabilized layer in situations with higher traffic demands. This is illustrated in Figure 31, which shows the source material for the bitumen stabilized layer grouped into two classes, and then plotted against the observed traffic that had been accommodated.

Figure 31 shows that there are no pavements in the LTPP knowledge base where natural gravel material was used as the source material for traffic situations in excess of 5 mesas. This observation is perhaps linked to the limited data set, and some of the HVS tests do suggest that stabilized natural gravel can accommodate traffic as high as 10 mesas. However, at this stage, based on the trends observed in the LTPP dataset, and considering the lack of detailed information on material type, it is recommended that the use of natural gravel source material for bitumen stabilized layers be limited to situations where the design traffic is less than 7 mesas.

A key observation in support of the above recommendation is that there are only three pavements in the dataset which are considered to have failed prematurely. In all three these cases, the bitumen stabilized layer involved a natural gravel.

![Figure 31: Source Material Quality versus Traffic Accommodated](image-url)

8.5 PROBLEM MATERIALS AND CONSTRUCTION PRACTICES

Discussions held with experienced practitioners during the course of this study suggest that certain materials are more prone to exhibit premature failure or problems during construction. One material type that falls into this category is weathered dolerite. However, it is likely that there are other materials, like dolerite, which are prone to present special problems when used with bitumen stabilization. It is strongly

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♣ It should also be noted that this study deals only with structural design. The design of the bitumen stabilized material is addressed in a separate study which is currently still in progress. It should be noted at this stage that appropriate specifications for strength and durability parameters for different design traffic scenarios will be developed and implemented as part of the final design guidelines. The outcome of the materials design component of this study may to some extent reduce the recommended constraint on source material use for high traffic applications.
recommended that – as part of the pilot testing of the proposed design method – a list of such materials be compiled based on interviews with experienced practitioners, and that this list be included in the finalized design guidelines to warn designers of potential risks at the design stage.

Discussions with experienced practitioners also showed that some construction practices introduce risk of premature failure. One example is half-width construction where the difficulty in obtaining equal levels at the longitudinal construction joint results in a step at the joint, which is difficult to correct. As with problem materials, it is recommended that a list of problematic or risky construction practices be compiled as part of the project to compile guidelines for construction. This list – with measures to compensate for it – should then be included in the finalized design guidelines.

8.6 **A System to Quantify Overall Risk**

It was noted in Section 8.2 that the risk of premature failure is increased when different facets of the design-and-construction process are executed without considering the combined risk introduced by small risk factors in separate parts of the process. This issue can be addressed mainly by educating designers about the key risk factors. One way to do this is by introducing a risk assessment system.

Table 14 shows a conceptual example of such a risk assessment system. The example shown in Table 14 is based on the system developed by Creagh (2005), and was expanded to include aspects that are considered to be especially relevant for applications in southern Africa. Naturally, it may not be possible for the designer to assess all aspects of such a scheme at the pavement design stage. However, the scheme makes designers aware of risk factors, and provides one way of ensuring that appropriate specifications are implemented. Such a scheme, if properly calibrated, will make it possible for a designer to monitor the accumulated risk throughout the design-and-construction process.

It is important to note that the scheme shown in Table 14 is conceptual only. Before it can be implemented, the system will need to be refined and a calibrated risk scoring system will need to be developed. This refinement and development of a scoring system can be done through consultation with experienced practitioners, and it is recommended that this be done as part of the pilot implementation process, working from the basis provided by Table 14. It is further recommended that the calibrated and refined risk assessment system be implemented as part of the final design guidelines.
### Table 14: Example of an Overall Risk Management System for Pavements Involving Bitumen Stabilized Layers

<table>
<thead>
<tr>
<th>Category</th>
<th>Risk Factor</th>
<th>Very Low Risk</th>
<th>Low Risk</th>
<th>Medium Risk</th>
<th>High Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pavement Design</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>What road category is being designed for?</td>
<td>Category A</td>
<td>Category B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Where does the design traffic lie on the range of expected traffic?</td>
<td>At or above upper limit</td>
<td>Upper quarter of range</td>
<td>Third quarter of range</td>
<td>At or below middle of range</td>
<td></td>
</tr>
<tr>
<td>How was overloading accounted for in the design traffic calculations?</td>
<td>Special extra analysis</td>
<td>Implicit calculations</td>
<td>Not taken into account</td>
<td></td>
<td></td>
</tr>
<tr>
<td>How was traffic information obtained?</td>
<td>Surveyed for this project</td>
<td>From recent survey</td>
<td>PMS Data</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Has a similar design been used with success for this traffic demand?</td>
<td>Definitely</td>
<td>Possibly</td>
<td>Don't know</td>
<td>New technique</td>
<td></td>
</tr>
<tr>
<td>Is there a risk of the road being flooded?</td>
<td>None</td>
<td>Unlikely</td>
<td>Small risk</td>
<td>Definite Risk</td>
<td></td>
</tr>
<tr>
<td>Are there paved shoulders?</td>
<td>Yes, in all areas</td>
<td>In some areas</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parts of the road situated on low embankments (&lt; 300 mm above terrain)</td>
<td>None</td>
<td>Small percentage</td>
<td>Large Percentage</td>
<td>Mostly</td>
<td></td>
</tr>
<tr>
<td>Parts of the road situated over wetland area</td>
<td>None</td>
<td>Small percentage</td>
<td>Large Percentage</td>
<td>Mostly</td>
<td></td>
</tr>
<tr>
<td>Relative quality of drainage</td>
<td>Highest standard</td>
<td>Acceptable</td>
<td>Marginal</td>
<td>Substandard</td>
<td></td>
</tr>
<tr>
<td><strong>Materials: Coarse Aggregate</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate mechanical strength</td>
<td>Very High</td>
<td>High</td>
<td>Fair</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate hardness</td>
<td>Very High</td>
<td>High</td>
<td>Fair</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate durability</td>
<td>Very High</td>
<td>High</td>
<td>Fair</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>Percentage of uncrushed coarse aggregate</td>
<td>0 to 5%</td>
<td>5 to 15%</td>
<td>15 to 40%</td>
<td>&gt; 40%</td>
<td></td>
</tr>
<tr>
<td>Likelihood of undesirable impurities</td>
<td>None</td>
<td>Can occur</td>
<td>Likely</td>
<td>Certain</td>
<td></td>
</tr>
<tr>
<td><strong>Materials: Fines</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plasticity Index of material passing the 0.075 mm sieve</td>
<td>&lt; 4</td>
<td>4 to 8</td>
<td>8 to 12</td>
<td>&gt;12</td>
<td></td>
</tr>
<tr>
<td>Percentage of material passing the 0.075 sieve</td>
<td>5 to 9%</td>
<td>3 to 5% or 10 to 12%</td>
<td>0 to 3% or 12 to 15%</td>
<td>&gt;15%</td>
<td></td>
</tr>
<tr>
<td>Risk of expansive fines</td>
<td>None</td>
<td>Can occur</td>
<td>Likely</td>
<td>Certain</td>
<td></td>
</tr>
<tr>
<td>Likelihood of a high percentage of non-cohesive alluvial fines</td>
<td>None</td>
<td>Can occur</td>
<td>Likely</td>
<td>Certain</td>
<td></td>
</tr>
<tr>
<td>Prevalent location of grading within the specified grading limits</td>
<td>Middle of envelope</td>
<td>Coarse side of envelope</td>
<td>Fine side of envelope</td>
<td>Sometimes outside envelope</td>
<td></td>
</tr>
<tr>
<td><strong>Materials: Mix Design</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Variability of material sources and material quality</td>
<td>Single controlled source</td>
<td>Single uncontrolled source</td>
<td>Possibly two sources</td>
<td>To or more sources</td>
<td></td>
</tr>
<tr>
<td>Is the source material one of those listed as being problematic?</td>
<td>No, not all</td>
<td>Possibly</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>How many mix designs has designer completed (HMA and BSM materials)?</td>
<td>More than five</td>
<td>Two to five</td>
<td>One</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>Sophistication of mix design process (HMA and BSM Materials)</td>
<td>Performance Based Tests</td>
<td>Indicator Tests</td>
<td>Mainly experience</td>
<td>No mix design</td>
<td></td>
</tr>
<tr>
<td><strong>Construction</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>How many projects has the contractor completed using this technique?</td>
<td>More than five</td>
<td>Two to five</td>
<td>One</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>How many projects has the Site Agent completed using this technique?</td>
<td>More than five</td>
<td>Two to five</td>
<td>One</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>What is the relative quality of site testing procedures and facilities?</td>
<td>Highest standard</td>
<td>Acceptable</td>
<td>Questionable</td>
<td>Infrequent testing</td>
<td></td>
</tr>
<tr>
<td>Will the contractor be allowed to construct trial sections?</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>How often will the Resident Engineer be on site?</td>
<td>Permanently</td>
<td>Part-time</td>
<td></td>
<td></td>
<td>No Resident Engineer</td>
</tr>
<tr>
<td>How many projects of this nature has the Resident Engineer worked on?</td>
<td>More than five</td>
<td>Two to five</td>
<td>One</td>
<td>None/No Resident Engineer</td>
<td></td>
</tr>
<tr>
<td>What is the likelihood of heavy rain during the construction period?</td>
<td>Very Unlikely</td>
<td>Unlikely</td>
<td>Likely</td>
<td>Certain</td>
<td></td>
</tr>
<tr>
<td>Are there permeable or semi-permeable surfacings involved?</td>
<td>No</td>
<td></td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Will there be an incentive to achieve highest possible density?</td>
<td>Yes</td>
<td></td>
<td>No</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
9 SUMMARY AND RECOMMENDATIONS

This report presents an outline of, and detailed methodology for, a structural design method for pavements that incorporate bitumen stabilized materials. The method relies on an index, called a Pavement Number (PN) to quantify the long term load spreading capacity of the pavement system. The PN calculation method was designed to ensure that basic principles of pavement behaviour and performance are incorporated. As such, the method is essentially a knowledge based, or heuristic, design method that relies on established rules of thumb to guide the design process.

The relationship between PN and pavement structural capacity was calibrated using a database of pavements for which the structural capacity was known with some certainty. This database was also used to develop criteria, by means of which the calibrated PN can be used to determine the appropriate design capacity of a pavement structure. Key aspects of the development are summarized below.

9.1 SUMMARY OF THE PROPOSED METHOD

9.1.1 Motivation for Adopted Approach

The use of a pavement index (i.e. the PN) to determine structural capacity presents a significant departure from other recent pavement design developments, which generally follow the Mechanistic-Empirical (ME) approach. There are five reasons why an index-based approach was adopted instead of the ME approach:

1. **Ease and transparency of calibration and validation**: compared to the ME approach, the index based approach is easier to calibrate and validate, especially when the limitations of available behaviour and performance data are taken into account.

2. **Unambiguous inputs**: the inputs for the method are material classes, which are relatively easy to determine consistently from data collected in routine pavement condition surveys. By contrast, the ME approach relies on the ill-defined resilient modulus, which is difficult to determine consistently as part of routine investigations (especially by inexperienced designers).

3. **Robustness**: the index based approach, as developed here, is believed to be more robust than the ME approach. As such, it is more suited to less experienced designers, and is also less likely to result in impractical or inappropriate designs.

4. **Better linkage to specifications**: the main inputs for the proposed approach consist of material classes, which are directly linked to established specifications. As such, the proposed method is tightly coupled to specifications, thereby closing the gap between design and construction processes.

5. **Transparency and educational value**: the proposed method can be implemented by means of a spreadsheet or paper worksheet. Each step of the calculation process relies on established principles of pavement design, and the transparency of the method is instructive and educational in its own right.

The disadvantages of the index based approach were discussed. It was noted that three main deficiencies of this approach were:

1. **Non-uniqueness of the pavement index**: traditional pavement indices like the AASHTO Structural Number (SN) are insensitive to the position of layers within the pavement system. Thus if the base and subbase were switched, the pavement index would remain the same.

2. **Insensitivity to weak layers**: unlike the ME design method, which focuses on individual layers, traditional index-based methods focus on the overall load spreading of the pavement. Because of this, most traditional index-based methods cannot identify weak zones within the pavement.

3. **Limited analytical ability**: index based methods provide a robust and coarse indication of expected pavement performance. As such, they do not have the same analytical capability as ME based design methods.

These disadvantages were discussed, and it was noted that the first two weaknesses can be addressed by incorporating basic design principles in the calculation of the Pavement Number. Furthermore, the
knowledge based calculation process can be combined with local knowledge of the performance of specific materials. This knowledge can be incorporated in the method in the form of design rules which would effectively disqualify inappropriate or high-risk design solutions.

It was acknowledged that the PN based design method does not have the same analytical strength as the ME design method. However, it is believed that the proposed method is better suited to practitioners with low to intermediate levels of experience. As such, the method is more appropriate to the roads-building industry of southern Africa, which is currently faced with a depleted and relatively inexperienced workforce.

9.1.2 Points of Departure

The rules-of-thumb which were used as points of departure for the PN-based design model were discussed, as were the key concepts needed to quantify these rules in a heuristically determined PN. Key concepts included in the method are:

- In the PN model, the effective long term load spreading capacity of a layer is represented by a model parameter known as the effective long term stiffness (ELTS).
- The calculation of the ELTS for a specific layer depends on the material type and on the situation in which the layer is placed. The stiffness of the supporting layer is of particular importance in determining the ELTS.
- The ELTS of the subgrade is the starting point for design, and greatly influences the relative stiffness of the overlying pavement system. In the PN model, the ELTS of the subgrade is determined by the material class, the climate and by the depth of cover over the subgrade.
- The general method for determining the ELTS relies on the modular ratio limit and the maximum allowable stiffness. For these parameters, different values are assigned to different material types and were calibrated using the available knowledge base of pavement structural capacity.
- The ELTS of a layer is determined as the minimum of (a) the support stiffness multiplied by the material’s modular ratio limit; and (b) the maximum allowable stiffness assigned to the material type.
- The modular ratio concept is also applied to highly cohesive materials such as cement stabilized materials and hot mix asphalt. In these cases, the modular ratio serves to quantify the influence of support stiffness in determining the rate at which such materials will break down due to fatigue.
- In the case of cement stabilized materials, the layer thickness is also used to adjust the ELTS. This adjustment mimics the effect that layer thickness has on the rate of crack propagation in these materials.
- Bitumen stabilized materials are assumed to behave in a manner similar to crushed stone material, but with a higher cohesive strength, which is simulated by the assignment of a higher modular ratio limit.
- A Base Confidence Factor is incorporated in the PN model to ensure that inappropriate base types are not used in wet climates or in high traffic situations.

The method for calculating the Pavement Number was detailed, and calibrated constants and relationships required as part of the calculation were documented.
9.1.3 Long Term Behaviour of Bitumen Stabilized Materials

The database of LTPP sections involving bitumen stabilized materials was analyzed to assess the long term behaviour of bitumen stabilized materials. This database includes, for some LTPP sections, behaviour indicators recorded at different stages of the pavement’s service life. These indicators include parameters such as DCP penetration rate, material state (loose or bound) and deflections.

The analysis of behaviour indicators suggests that bitumen stabilized materials with low binder contents behave more like stiff - yet flexible - unbound materials than stiff and brittle cement stabilized materials. A behaviour model for bitumen stabilized materials with low binder contents was postulated, based on the available evidence. Key aspects of the model are:

- The behaviour of bitumen stabilized materials involving low binder contents is believed to be similar to that of an unbound granular material, but with a significantly improved cohesive strength. This behaviour places bitumen stabilized materials somewhere between unbound granular materials and intact cement stabilized materials. The material behaviour will lean toward the latter when excessive cement is added.

- Bitumen stabilized materials perform well when cohesive strength is optimized through proper mix design (to determine the optimal binder and cement contents), whilst retaining enough flexibility so that friction resistance is still activated during loading.

- Similar to unbound granular materials, the stiffness of bitumen stabilized materials is dependent on the stiffness of the support. However, the high cohesive strength allows the material to sustain a higher modular ratio under loading (when compared to unbound granular materials).

- The assumed maximum allowable stiffness of the materials were based on the observed behaviour in the LTPP sections. However, the suitability of the assumed value may change significantly depending on the relative amounts of bitumen and cement added.

The above principles are assumed to hold for balanced mix designs with well-graded source materials. Optimal shear strength of the material can be compromised in two ways:

- Excessive amounts of cement will transform the material from a somewhat flexible to a brittle state. In this state, the cohesive strength will dominate but will significantly reduce once fracture occurs. This is likely to be associated with deformation and cracking, and will result in a material consisting of large, fractured clumps, with a low frictional resistance.

- Poorly graded or non-durable source materials (e.g. soft weathered natural gravel or material with excessive fines) will compromise the frictional resistance of the material. Inexperienced designers may be tempted to compensate for such a situation through the addition of higher amounts of cement. Such fine grained, brittle material will be highly susceptible to crushing and fatigue failure.

9.1.4 Calibration and Validation

Constants and relationships in the PN model were calibrated using three data sets. These data sets consisted of: (a) structures from the TRH4 catalogue; (b) structures in the LTPP database which had carried enough traffic to facilitate an assessment of their likely structural capacity; and (c) structures tested with the Heavy Vehicle Simulator (HVS), and for which the structural capacity can be estimated based on applied load and pavement condition.

The calibrated model correlated well with the observed structural capacity (or allowed structural capacity in the case of the TRH4 dataset). The best correlation between PN and structural capacity was observed in the case of the LTPP dataset, and for pavements that are currently in a warning condition. It is believed that the calibrated PN model exhibits a suitably reliable relationship between the pavement structure and its observed structural capacity to facilitate use of the model for design purposes.

Design criteria were developed using the three available data sets. The criteria allow the determination of a pavement’s design capacity based on its PN value. The design criteria involve a step function which
was fitted to the data to ensure optimal assignment of structural capacity for the pavements in the three available data sets. The criteria were then validated by comparing the PN-assigned design capacity of pavements in the LTPP and HVS data sets with the actual observed performance of these pavements. It is believed that the model provided an appropriate assessment of the pavement capacity in almost all instances.

9.1.5 Practical and Peripheral Aspects

Peripheral aspects which relate to the reliability of bitumen stabilized pavements were discussed. Key elements here are the surfacing type and source material used. Almost all pavements in the LTPP dataset were constructed using asphalt surfacing (as opposed to only a surface seal). It is believed that recycled materials (consisting of stabilized crushed stone or natural gravel), do not have the same crushing strength and hardness as the original source material. It is therefore recommended that where possible a thin asphalt surfacing be used for situations where the design traffic is greater than 1 million equivalent standard axles (mesa). For design traffic in excess of 10 mesa, a surfacing thickness of 50 mm is recommended.

For the majority of pavements in the LTPP and HVS data sets, the bitumen stabilized layer incorporated source material consisting of reworked crushed stone, cement stabilized crushed stone or reclaimed asphalt pavement (RAP). There are relatively few cases where the bitumen stabilized layer consisted of natural gravel. Where natural gravel was used, the traffic intensity was generally low, with observed accumulated traffic of 5 mesa or less.

Compared to bitumen stabilized crushed stone, stabilized natural gravel seems to exhibit a greater risk of premature failure. Three pavements in the LTPP dataset are believed to have exhibited premature failure, and all three of these used natural gravel as a source material.

In view of these observations, the use of bitumen stabilized natural gravel is at this stage not recommended for situations where the design traffic is greater than 7 mesa. This constraint may not apply in cases where high strength material is involved, and in which a thorough mix design is performed to ensure that the material has adequate stability and durability. This constraint should be reconsidered once the findings of the mix design component of the study have been completed.

It was noted that risk of premature failure increases when marginal materials or risky assumptions enter into more than one phase of the design-and-construction process. Designers should take a more comprehensive view of the design-and-construction process, and should be made aware of all possible risk factors at the design stage, so that these factors can be monitored during subsequent phases (i.e. development of specifications, mix design and construction). To facilitate this, a simple risk management system was proposed in concept, and recommendations were made for the refinement of such a system to enable its incorporation in the final design guidelines.

9.2 Recommendations

- It is recommended that the proposed design method for bitumen stabilized pavements be pilot tested during 2007. Selected practitioners should be trained in the use of the method to enable an assessment of the design outcomes in routine design situations. These assessments should focus on the suitability and reasonableness of the method outcomes, and on robustness. Feedback should be used to refine the method and make it more robust for general use.

- The refined method should be used to develop a design catalogue for bitumen stabilized pavements. This catalogue should focus on situations where the design traffic is less than 1 mesa, and should incorporate practical considerations for these situations.

- It is believed that some materials, like weathered dolerite, present special challenges and risks when used with bitumen stabilization. It is recommended that more such problem materials be identified as part of the pilot testing phase, and in conjunction with the mix design component of this study which is currently in progress. The finalized guidelines should list all identified problem materials, and should provide measures for mitigating the risks when these materials are involved.

- A simple risk assessment system was proposed to integrate risk factors that enter at various stages of the design-and-construction process. It is recommended that this system be expanded and refined to facilitate its inclusion in the finalized guidelines.
10  REFERENCES


APPENDIX A:  PROJECT WORK PROPOSAL
Project Proposal Number:  PP/ 2005/ 09/ d

Version: 1.0

Updating Bituminous Stabilized Materials Guidelines, Phase 2: Development and Calibration of Structural Design Procedure

Submitted by:  Dr Fenella Long, MAS

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</table>

1. TERMS OF REFERENCE

This proposal forms part of a larger study proposal, entitled “Compilation of a Bituminous Stabilized Materials Guideline Document for Foamed and Emulsified Bitumen Treated Materials” (Proposal Number PP/2005/09). As such, the tasks and methodology defined in this proposal comprise a sub-task of a larger project and should be viewed as such. The background, project objectives, expected benefits and implementation plans are described in detail in proposal PP/2005/09, and will not be restated here. Briefly, the project pertains to the updating of the TG2 and TG(X) guidelines, which are intended to guide and assist practitioners in the selection and design of pavements and materials that incorporate bituminous stabilization consisting of bitumen emulsion or foamed bitumen. The objectives of the larger project are to improve or redesign the modules relating to the mix design and structural design, to ensure that the guidelines reflect the latest best practice as well as all available research and field performance data. The overriding objective is to compile a complete guideline document incorporating the mix and structural design and construction guidelines.

The first phase of the project was an inception study with two components, mix and structural design. An objective of the inception study was to plan further testing and development activities needed for the thorough revision of the TG2 and TG(X) guidelines. It was recommended that in Phase 2 of the project the mix design and structural design guidelines be developed and reviewed, and in Phase 3 the guideline document be compiled.

This proposal pertains to the structural design component of Phase 2 of the larger project, which comprises the development of the structural design guidelines. The development of the mix design guidelines is detailed in a companion proposal (PP/2006/03/c). The two components of Phase 2 should be executed in parallel.

2. PROJECT OBJECTIVES

This structural design guideline development project has three objectives:

1) To expand the LTPP database compiled during the Inception Study to a sufficient level to enable the development of classification based design guidelines.

2) Develop and populate a classification based design matrix.

3) Peer review of the recommended design guidelines.
3. METHODOLOGY

The methodology proposed in the following paragraphs is in accordance with the framework for a structural design method as documented in the Structural Design Inception Study Reports:


This development of the structural design guidelines for a Bituminous Materials Guideline will comprise of the following tasks:

Task 1: Update LTPP Summaries with Recently Available Information
At the time of the inception study some information on the LTPP sections studied was not available (e.g. the SANRAL 2005 network survey and the current condition). In this task, this information will be accessed and the LTPP summaries updated. It may not be possible to obtain actual performance data on the current condition, however, and in those cases the clients or relevant engineers will be contacted to obtain a subjective opinion of the current condition.

Task 2: Peer Review of LTPP and HVS summaries
Since the LTPP and HVS summaries developed in the Inception Study will form the empirical base for the structural design methodology, it is essential that these summaries be accurate and appropriate. This task therefore involves a peer review of the LTPP and HVS summaries developed in the Inception Study. The project engineers, resident engineers and clients originally involved in the design and construction of each section will be contacted to:

- Validate and/or correct the information contained in the LTPP summaries, and
- Expand the summaries to include practical hints and tips that ensured successful construction.

The peer review will also be performed on any additional LTPP summaries compiled in Task 5.

Task 3: Develop Material Classification Method
This task involves developing a robust, appropriate method for a classification based design method. The following specific subtasks are involved:

- Study and select model for determining the appropriate material class from several indicators.
- Develop and test the selected model.
- Identify and contact practitioners, and hold workshop to select classification parameters and ranges.

This task will be executed in conjunction with the mix design project team, as the mix design output will form important inputs into the material classification system. However, the theoretical framework of the method can be developed prior to the availability of the recommended mix design method with the associated tests and classification limits. The method will then be refined in Task 7 when the mix design results are available and any additional LTPP data have been collected.

Task 4: Develop Structural Design Matrix/Method
In this task the design matrix will be populated with the available data. This requires:

- An in-depth study of the available LTPP and HVS data.
- Formulation of the structure of the design matrix.
- Assignment of the pavement structures to the design matrix.
- Interpolation of new structures using mechanistic principles for unpopulated areas within the design matrix.

Task 5: Expand LTPP Database
In this task, the database of LTPP pavements developed in the Inception Study will be expanded to address the deficiencies identified in the Inception Study and from the previous task. At this stage it
is envisaged that an additional 10 to 16 sections will need to be included, however this will be finalised only after completion of Task 4.

The data gathered in this task will include the following, where applicable:
- Identification and assessment of failed pavements;
- Additional foamed bitumen pavements;
- Pavements with natural gravel subbases (particularly with thick recycled bases);
- Pavements on poor quality subgrades;
- Foam pavements constructed on stabilised subbases, and
- BSM pavements constructed as part of a newly constructed road.

The data will be obtained by three means. Firstly, analysis of existing as-built records, available behaviour and performance data and traffic assessments in line with the methodology used to prepare LTPP summaries in the Inception Study. Secondly, some sections will require field measurements, as discussed in the next task. Thirdly, some sections, such as the sections on poor quality subgrades will be developed by interpolating between, or extrapolating from, existing structures using mechanistic principles.

Task 6: Collect Additional Field Data
This task involves the collection of field data from in-service pavements. This is only required for pavement types where no data are already available, and will be limited to the collection of essential data in an effort to limit the total project cost. Some or all of the following data will be collected:
- Testpits and material tests
- Deflection measurements (FWD)
- Visual condition
- Rutting measurements
- DCP tests
- Roughness measurements
- Core extraction

The data will be used to compile LTPP summaries for the sections in line with the summaries prepared in the Inception Study.

Task 7: Refine Material Classification and Design Method
The material classification and design method developed in Tasks 3 and 4 will be refined with the data collected in Tasks 5 and 6 and on inclusion of the recommended mix design method selected in the companion Phase 2 project. This task can therefore only begin when the mix design method is finalised.

The populated design matrix will also be refined and validated through liaison with identified expert practitioners.

Task 8: Simple Methodology for Designing Structures not Included in the Design Matrix (optional)
In this task, the method used to design the non-validated structures in the design matrix will be expanded and documented. This will be done by formalising the method and inputs used and defining the inputs required. This method should only be used by experienced practitioners, and could be included in the final BSM guideline as an appendix. Note that this task involves the use of mechanistic principles, and not the mechanistic-empirical method. As such, transfer functions will not be used.

This task is optional.

Task 9: Strategy for On-going Population of the Design Matrix
This task involves the development of a strategy to facilitate the on-going population of the design matrix as data on more pavements become available, or as updated data on pavements included in the design matrix become available. This involves determining a strategy for alerting practitioners
and road authorities to the need for more data, and the compilation of a detailed list of both the required and desired data.

Task 10: Documentation

The project findings will be detailed in a technical memorandum, which will serve as a backup to the guidelines to be presented in the guideline document. The findings will also be summarized in a project summary report.

4. DELIVERABLES

The deliverables for this project will consist of a technical memorandum, a project summary report and a detailed presentation to the project funders. The project documentation will contain the following information:

- Expanded LTPP summaries;
- Description of method selected for material classification from several indicators;
- Populated design matrix;
- Method for designing structures not explicitly included in design matrix (optional), and
- Strategy for obtaining new data on pavements as it becomes available.

The project documentation will be structured according to the Gautrans Documentation Guidelines so that raw data and technical discussions are limited to the technical memorandum, while the project summary report will concisely summarize key observations and recommendations.

5. SKILLS DEVELOPMENT

The skills development part of this project will involve the use and associated mentoring of a Gautrans technician or young engineer in the project work, particularly in Tasks 5 and 6 as outlined in the Methodology section. The skills imparted will be:

- Data retrieval
- Analysis of retrieved data
- Exposure to project documentation, such as as-built records, design reports and Pavement Management Systems
- Basic reporting

The cost of employment of the mentee will be carried by Gautrans as part of their normal salary. The cost of the mentoring by the project team is included in the Project Cost section of this proposal.

6. PROJECT PLAN

6.1. Project Team/Personnel

The project team will consist of Dr Fritz Jooste, Dr Fenella Long and Sanet Jooste of MAS with Prof. Kim Jenkins acting in an advisory role. Details of the project team are summarized below.

<table>
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<tr>
<th>Name</th>
<th>Organisation</th>
<th>Contact details</th>
<th>Hourly Rate</th>
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<tbody>
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<td>R 500</td>
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The development of the mix design guidelines will be championed by Professor Kim Jenkins. The output from the mix design tests are an input into the structural design guidelines, and therefore the mix design and structural design teams will work in tandem to ensure the mix and structural design guidelines are aligned.
6.2. Project Costs

Detailed costs are only provided for Phase 2: Structural Design Procedure Development. These costs are detailed in Table 2. The cost of Task 6 is a range, which depends on the number of field sites from which field data will be collected. The number of sections will be finalised after the completion of Task 4 and in consultation with the clients. The work will be done by Fritz Jooste, Fenella Long and Sanet Jooste. Two workshops with experienced practitioners will be held during the project. The costs shown include the time cost of the project team, but do not include the cost of holding the workshops. The Project Manager’s costs are not included in this proposal, but are listed in the master proposal.

Table 2. Project Costs for Structural Design Procedure Development

<table>
<thead>
<tr>
<th>Task Type</th>
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<th>Detailed Action</th>
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<th>Cost Minimum - Maximum</th>
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<tr>
<td>Data gathering tasks</td>
<td>1</td>
<td>Update LTPP summaries with recently available information</td>
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<td></td>
<td>2</td>
<td>Peer review LTPP and HVS summaries</td>
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<td></td>
<td>5</td>
<td>Expand LTPP database</td>
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<td></td>
<td>6</td>
<td>Collect additional field data</td>
<td>(3 to 10 sites at R 50 000 per site)</td>
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<tr>
<td>Development tasks</td>
<td>3</td>
<td>Develop material classification method</td>
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<td></td>
<td>4</td>
<td>Develop structural design matrix/method</td>
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<td></td>
<td>7</td>
<td>Refine material classification and design method</td>
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<td></td>
<td>8</td>
<td>Simple methodology for designing structures not included in design matrix</td>
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<td>Reporting tasks</td>
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<td>Strategy for on-going population of design matrix</td>
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<td>Other</td>
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<tr>
<th></th>
<th>Sub Total</th>
<th>Value Added Tax (14 %)</th>
<th>Total</th>
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* This task is optional

6.3. Time Line

The Gantt chart is shown in Figure 1. The project will take 12 months to complete, however the start of some tasks is dependant on the completion of previous tasks and the Mix Design study. The estimates shown in Figure 1 are based on sufficient information coming from Part 1 of the applicable Mix Design Tasks (see Phase 2 Mix Design Proposal). Should this not be the case, the completion of the tasks shown will be delayed.

Figure 1. Gantt Chart