INTERIM GUIDELINES FOR THE DESIGN OF HOT-MIX ASPHALT IN SOUTH AFRICA

Prepared as part of the Hot-Mix Asphalt Design Project

September 2001

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Overview of the Design Guidelines

Background

The Hot-mix Asphalt (HMA) design method described in TRH8:1987 has been used in South Africa for over a decade. TRH8:1987 is centred on the Marshall design method, but includes additional information and criteria for component evaluation. Over the past decade, several changes have taken place in the road building industry, which have exposed deficiencies in the scope and depth of the methodology contained in TRH8:1987. These changes include:

- more aggressive design situations caused by increases in legal axle weight limits and heavy traffic volumes;
- influx of overseas information and of new methods which may lead to a fragmentation of methods used in South Africa, and
- increased use of mixes such as Stone Mastic Asphalt (SMA) and Large Aggregate Mix Bases (LAMBS), for which no adequate provision is made in the TRH8:1987 document.

In view of the deficiencies in the existing design approach, practitioners in the field of HMA design, through committees such as the South African Bituminous Materials Liaison Committee (BMLC), have acknowledged that improvements in the design of hot-mix asphalt have become a necessity. To this end, a project was launched in early 1998 with the aim of developing a new HMA design method which incorporates state of the art knowledge of materials evaluation, mix design and performance assessment, and which takes cognisance of climatic and pavement environments as well as of aspects related to construction.

This interim guideline document is a preliminary product of this project and is to be used in parallel with TRH8: 1987 to validate the proposals and develop criteria for future implementation.

Purpose

The HMA design guidelines are intended to:

- provide background information for consideration during selection of the mix type as well as in the selection of the most appropriate performance tests for any design situation;
- provide designers with information that pertain to climate, pavement structure and aspects of construction such as materials availability, and which may have an impact on mix selection and design;
- provide designers with basic information on the selection and evaluation of aggregates, binders and fillers;
- introduce new approaches to volumetric design and to the selection of optimum binder contents for different types of mixes, and
- introduce new approaches to performance testing.

In their present form, the guidelines are not intended to serve as a mix design manual. That is, they do not provide a step-by-step formulation of the design of different mix types. Rather, they outline the methodology and most important considerations to be made during the design stage. The guidelines are also not intended to replace the current TRH8:1987 document. However, it is hoped that, in time, the design guidelines will evolve into a document which may be specific enough to replace the current TRH8: 1987 document.
Structure and Scope

Figure 1 shows the structure of the generic design procedure contained in the guidelines, and also indicates the section of the guidelines in which each task is discussed. The design process can roughly be divided into four phases:

i) Preliminary considerations leading to mix selection and rating of design objectives;
ii) Component evaluation (aggregate, binder and filler);
iii) Volumetric design, leading to the selection of gradation and an optimum binder content, and
iv) Performance testing.

Phases (i), (ii) and (iv) are general and can be applied to any mix type. Phase (iii), volumetric design, is subdivided into different subsections. The first subsection (Section 4.1) deals with the basic steps and objectives to be reached during volumetric design. The following subsections (Sections 4.2 to 4.6) deal with specific mix types. In their present form, the design guidelines contain information on the following mix types:

1. Dense-graded mixes;
2. Stone mastic asphalt;
3. Open-graded Mixes, and
4. Large Aggregate Mixes for Bases.
Preliminary Considerations and Rating of Design Objectives

This is the first task a designer should undertake when designing an HMA mix for a specific application or environment. Chapter 2 of the design guidelines contains a discussion of the elements that may influence the selection of mix type, as well as considerations relating to construction and to the availability of components which may have an impact on project specifications. Briefly, these considerations are:

i) Traffic considerations and definition of traffic intensity;
ii) Pavement considerations (evaluation of support conditions and their impact on mix selection and design of the asphalt base and/or asphalt wearing course);
iii) Climate;
iv) HMA Layer thickness considerations, and
v) Other considerations (availability of materials, construction issues, pavement geometry etc.).

Proper consideration of all these issues allows designers to evaluate the different design objectives (stability, fatigue resistance, environmental durability and permeability) in terms of relative importance. This information can be used to select the most appropriate mix type, as well as the level of performance testing needed for a specific project. Table 2.6 and the discussion in Section 2.8 provide designers with information on the advantages and disadvantages of different binders and mix types.

Component Evaluation

The evaluation of the different mix components (aggregate, filler and binder) is discussed in Chapter 3. Table 3.1 provides a summary of the tests and criteria required for the evaluation of aggregate properties such as hardness, durability, surface texture and cleanliness. Filler properties and issues related to binder-filler combinations are discussed in Section 3.3. Binder properties and elements to be considered in the evaluation of binders (including modified binders) are discussed in Section 3.4.

Volumetric Design

The volumetric design process, as described in Section 4.1, consists of five steps:

i) Spatial considerations, in which the designer has to evaluate the basic structure of the HMA mix, and the manner in which stability will be achieved. Appendix B provides a more detailed discussion of spatial design concepts.
ii) Selection of a target gradation;
iii) Sensitivity analysis, in which the sensitivity of volumetric parameters to small variations in different aggregate fractions is evaluated;
iv) Mixing and conditioning of samples using the recommended procedure for the Marshall method, and
v) Sample compaction and volumetric evaluation, in which the optimum binder content is determined.

If properly executed, these five steps should result in a design that is balanced in terms of stability, fatigue resistance, durability and permeability. To achieve such a balance, different types of mixes require different criteria and evaluation methods. Thus, the volumetric design process to be followed for different mix types is described in separate sections. Each section contains a basic description of the mix, how stability is achieved and where the mix should or should not be used. A flow chart is then used to describe the process for selecting the optimum binder content. Chapter 5 contains a summary of the process and parameters required for volumetric calculations.
Performance Testing

A proper volumetric design should result in a well-balanced mix that has adequate resistance to rutting, fatigue, ageing and water infiltration. For most design situations, however, some validation of the mix performance is required. Thus several performance evaluation tests are described in the design guidelines.

The level of performance testing depends on the ratings of the various design objectives. This applies particularly to evaluation of rutting and - to a lesser extent - of fatigue. For these aspects, the performance tests available differ in cost and sophistication. The guidelines provided in Chapter 2 allow designers to decide on the relative importance of the different design objectives. Chapters 6 and 7 contain discussions of the available methods for evaluation of rutting and fatigue potential, respectively. Chapter 8 contains a description of more general tests, such as those used to determine mix stiffness, stripping potential and permeability.

Validation and Further Developments

In their present form (September 2001), the interim design guidelines are intended to disseminate knowledge and to introduce new approaches to design and performance testing. Although many of the design aspects and test methods are well-known, some of the methods have only been recently developed and have not yet been well-validated in practice. An implementation phase has been planned in which the newly introduced methods and techniques will be tested and refined. Further work will also include validation studies to ensure that the design approach is conducive to high quality designs which will make optimal use of available funds and materials.

The first implementation phase was completed in September 2001. During this phase, the design guidelines were pilot tested (under the auspices of the HMA project management group) on several construction projects. The findings of future implementation phases will be used to refine and improve the current guideline document, so that this interim document can by replaced by a final draft during 2003.
1. PURPOSE AND STRUCTURE OF THE MIX DESIGN GUIDELINES

1.1 Background

The mix design guidelines contained in this document were developed to assist designers in the selection, design and evaluation of hot-mix asphalt (HMA). These guidelines stem from the hot-mix asphalt design project that was launched in 1997 as a partnering project managed and executed by public sector, consulting, contracting, educational and research organizations. The vision of the HMA design project was:

To develop a hot-mix asphalt design system which is integrated with pavement design, takes cognisance of issues related to environment and construction and which allows a rational evaluation of expected performance to be made. The method should incorporate the best local and overseas practice and technologies and should yield improved and appropriate asphalt designs for all design situations.

The mix design guidelines were largely formulated to serve this aim. As such, they are intended as a practical guide to designers of hot-mix asphalt, which will provide both broad and specific guidelines on:

- The selection of the most appropriate mix type through proper evaluation of traffic, climate, pavement structure, geometry and other physical restraints which may impact constructibility and mix performance;
- Selection and evaluation of mix components (aggregate, filler and binder);
- Mix design processes, both general and mix-specific, which take proper cognisance of spatial design concepts and which introduce new ways in which to evaluate mix performance and compactibility;
- Ranking of design objectives and selection of the most appropriate types of performance tests, and
- Basic design calculations and special considerations related to volumetric calculations.

Every effort has been made to develop or select guidelines, processes and test methods which are appropriate to South African design situations. In particular, an effort was made to retain the use of current equipment which is relatively inexpensive to obtain and operate. However, where appropriate, new methods of mix characterization have been introduced. These apply mainly to mixes which are intended for demanding climatic and traffic situations and for which new and more sophisticated test methods are needed in order to ensure that the mix is suitable for its intended application.

The criteria for the selection of mix components are closely linked to the recommendations contained in Technical Recommendations for Highways, Volume 14 (TRH14: currently under revision - the Standard Specifications for Road and Bridge Works prepared by the Committee of Land Transport Officials, COLTO, could be used in the interim). Also, some of the procedures and
test methods recommended for the design of HMA mixes are based on recommendations contained in TRH8:1987.

The present version of the HMA design guidelines is published as an interim document. The guidelines will be pilot-tested during the year 2001, after which the procedures will be re-evaluated and finalised. In time, the finalised guidelines should replace the current TRH8 document.

1.2 Scope

The guidelines cover the design of all types of hot-mix asphalt that are commonly used on road and airport construction projects in Southern Africa. These guidelines do not cover the use of cold mix materials or of mixes in which cut-back bitumens or emulsions are used.

1.3 Structure of the Guidelines

The manual is structured in the manner in which a typical design should proceed. Thus Chapter 2 deals with the selection of a mix type and the evaluation of the operating conditions. This chapter also provides information which can be used to link mix type selection and mix design to the pavement situation.

Chapter 3 describes the selection and evaluation of mix components and also describes the definition of volumetric design parameters.

Chapter 4 deals with the actual design process. This chapter describes the generic process to be followed during design and also outlines the general procedure for specimen compaction and conditioning. Chapter 4 is divided into several sections, each dealing with a specific mix type. The information contained in these subsections provide specific guidelines on the design of individual mix types.

Chapter 5 describes the calculations required to determine volumetric mix properties. This information is presented in a concise, step-wise format which can easily be programmed in a computer spreadsheet.

Chapters 6 and 7 provide an overview of tests and processes for the evaluation of rutting and fatigue, respectively.

Chapter 8 provides an overview of other tests related to the performance of HMA mixes.

1.4 Disclaimer

This guideline document should not be used as a substitute for knowledge and expertise. The concepts presented in this document should be applied by sufficiently experienced practitioners in engineering as a guide for the design of appropriate mixes.
2. RATING OF DESIGN OBJECTIVES AND MIX TYPE SELECTION

2.1 Introduction

This section describes the design issues which will have an impact on the selection of mix type as well as on the selection of tests and methods for design and evaluation of the mix. These issues are:

- Traffic considerations;
- Pavement considerations;
- Climate considerations;
- Construction issues;
- Other issues (construction, road geometry and materials availability) and
- Layer thickness considerations.

Each of these issues is discussed in the following sections. These discussions are then followed by recommendations for the selection of mix type and test methods, based on evaluation of the various design issues.

2.2 Traffic Considerations

The following traffic aspects play a role in mix selection and design:

- Number of heavy vehicles;
- Axle loads;
- Equivalency factors;
- Tyre pressures;
- Truck speeds;
- Wander across the width of the surface;
- Braking and shoving effects;
- Fuel spillage, and
- Light vehicle considerations.

While it would be ideal to quantify each of the above aspects in precise terms, current knowledge does not make this possible. The following paragraphs provide some general guidelines for use in HMA design to assist the designer in selecting and designing an appropriate asphalt mix.

Heavy Vehicles

The number of trucks and their axle loads need to be taken into account in the design of an asphalt mix. Pavements with large volumes of truck traffic require greater resistance to rutting, particularly if the underlying pavement is stiff, and to fatigue cracking, particularly if the underlying pavement is flexible.

The intensity of axle loads applied to the pavement will affect the resistance to permanent deformation of an asphalt layer because the mix gradually hardens over its lifetime. Early intense
loading, such as that which occurs on a rehabilitated pavement, is more severe than the less intense loading which normally occurs on a newly constructed road. Designers should also be aware that loading on newly laid asphalt may also cause premature problems.

For the purposes of mix design, heavy traffic intensity is evaluated through the use of traffic classes. The traffic classes used for mix design are defined in Section 2.3.

Axle Loads
Axle loads are limited to certain maximum values by law. Owing to the current lack of enforcement, there are a fairly large number of axle loads which exceed the value of 80 kN which is used as a standard in design calculations.

Equivalency Factors
Depending on the situation of the layer it may have a greater or lesser equivalency factor than that which is normally used for pavement design purposes. For example, equivalency values ranging from 4 to 10 have been reported for rutting in asphalt.

Tyre Pressures
Tyre configurations and tyre pressures play a significant role in rutting and fatigue cracking. The use of the super-single tyre is becoming prevalent in parts of Europe and its use in southern Africa is increasing. Also, if the tyre is under-inflated, the tyre wall will exert a significant pressure on the surface of the pavement. Based on current observations it would appear that tyre pressures of 900 kPa are not uncommon on southern African roads. Such high pressures place a greater stress on the asphalt layers (which are usually the upper pavement layers) and demand more stable mixes for high traffic conditions.

Traffic Speed
The speed of heavy vehicles may significantly influence the performance of an asphalt mix. At high speeds the impact of the load on the pavement system is resisted not only by the combined stiffness of the pavement layers, but also by the inertial and damping forces generated within the pavement structure. These resisting forces will increase with vehicle speed, with a resultant reduction in the amount of deflection and bending which takes place in the asphalt layer. Dynamic pavement models as well as strain measurements taken at various vehicle speeds have shown that tensile strains at the bottom of the asphalt layer may decrease by as much as 50 per cent as vehicle speeds increase from creep speed to about 80 km/h.

Lower vehicle speeds, on the other hand, influence rutting potential. At low speeds, the loading rate is significantly reduced. This effectively causes a decrease in the viscosity of the binder, and increases the demand for rutting resistance. Mixes designed for climbing lanes, intersections or any other condition where heavy vehicle speeds are predominantly less than approximately 30 km per hour require increased rutting resistance.

Wander
The degree of wander in the traffic lane can have a significant effect on rutting and fatigue. Wander is normally greater on lanes which are wide and have fast-moving traffic than on narrow lanes with slowly moving heavy traffic. In the latter situation the degree of channelization is increased and consequently greater rutting resistance is required.
Braking and Shoving
At intersections or steep upgrades braking and shoving forces can be significant, leading to increased rutting and shoving. This situation is aggravated by the low vehicle speeds associated with intersections or steep grades.

Fuel Spillage
Spillage of fuel, particularly diesel, can cause softening of the asphalt, leading to distress which may not be representative of the mix and which cannot be predicted at the design stage.

Light Vehicle Considerations
As far as mix stability and pavement design are concerned, it is primarily heavy vehicle considerations that need to be taken into account. The loads imposed by light vehicles (e.g. small passenger vehicles) are too light to induce damaging stresses and strains in a well-designed asphalt mix or pavement.

However, the volume and speed of light traffic need to be taken into account when functional properties such as skid resistance, noise reduction and riding quality are being considered. In general, high skid resistance is required for mixes placed on roads where the speed of light traffic will generally exceed 60 km/h. Mixes placed in urban areas, where the volumes of light traffic are high, may need to have improved noise reduction properties. Recommendations on the selection of mix type for improving skid resistance and noise reduction are made in Section 2.9.

2.3 Traffic Class

For the purposes of mix and pavement design, heavy traffic volumes are often placed in different categories, or traffic classes, to delineate different levels of traffic intensity. The traffic class affects the risk associated with the design and, hence, the level of testing required. It also affects the degree of compaction which is likely to occur under traffic during the early part of the design life of the asphalt and hence the volumetric criteria to be applied during design and construction. Table 2.1 shows the traffic classification for different volumes of heavy vehicle traffic.

For pavements in which the actual number of heavy vehicles is within 20 per cent of the upper limits shown in Table 2.1 for each class, the traffic class should be increased by one level if one or more of the following conditions apply:

- Large percentage of fully laden heavy vehicles (as, for example, on mine haul roads);
- Large percentage or potential of overladen vehicles, and
- Expected growth rate of heavy traffic greater than 10 per cent.
### Table 2.1 Traffic Classification

<table>
<thead>
<tr>
<th>Measure of Traffic Intensity</th>
<th>TRAFFIC CLASS</th>
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<tbody>
<tr>
<td>Number of Heavy Vehicles/ Lane/Day</td>
<td>Approximate Pavement Structural Design Capacity*</td>
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<tr>
<td>&lt;80</td>
<td>&lt; 1 million ESALs</td>
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<tr>
<td>80 to 200</td>
<td>1 to 3 million ESALs</td>
</tr>
<tr>
<td>200 to 700</td>
<td>3 to 10 million ESALs</td>
</tr>
<tr>
<td>&gt; 700</td>
<td>&gt; 10 million ESALs</td>
</tr>
</tbody>
</table>

* ESAL = Equivalent Single Axle Load. Traffic classes broadly correspond with TRH42 design classes.

### 2.4 Pavement Considerations

**General**

The pavement structure provides the support for the asphalt layer and as such comprises an important element to be considered in mix design. The interaction that takes place between the various pavement layers is complex and a description of even the more general behaviour patterns lies outside the scope of this guideline document. However, under most conditions the asphalt layer is primarily influenced by the properties of the layer immediately below it. This means that the interaction between the asphalt layer and the pavement system can be very adequately described if the properties of the base layer only (in the case of a thin asphalt surfacing) or of the subbase (in the case of a thick asphalt base) are taken into consideration. A possible exception to this would be where the support layer is thin (<100 mm).

Of the possible modes of distress of hot mix asphalt layers, only fatigue and (to a lesser extent) rutting, are influenced by the stiffness of the support layer, which - together with the applied load - determines the amount of bending that takes place in the asphalt layer. For an asphalt layer with a thickness of less than about 60 mm, this bending is determined almost completely by the stiffness of the support layer and only slightly influenced by the properties of the asphalt layer itself. When the asphalt layer is thicker than about 80 mm, the influence of the support layer decreases somewhat and the amount of bending taking place in the asphalt layer is partly determined by the properties of the asphalt layer itself.

The stiffness of the immediate support layer therefore to some extent determines the working stresses and strains under which the asphalt layer will operate. In the case of fatigue, the amount of bending taking place is critical for determination of the expected range of tensile strains in the asphalt. In the case of rutting, the support stiffness to some extent determines the amount of shear stress that is generated in the lower part of the asphalt layer. However, this influence can be overridden by factors such as temperature and tyre pressure. Furthermore, studies have indicated that the largest portion of rutting in asphalt layers originates in the upper part of the layer. For these reasons, it is often stated that rutting is primarily a mix design issue, while fatigue is both a pavement and a mix design problem.

Apart from the performance aspects, support conditions also need to be considered to ensure ease of construction of the HMA. Although compactibility of a mix is primarily affected by its
workability at paving temperatures, it is also significantly influenced by the stiffness of the base layer. In the case of thin asphalt layers, the final riding quality may also be determined to some extent by the properties and smoothness of the base layer. In the case of very weak or variable support conditions, the appropriateness of using an asphalt surfacing should be questioned.

Stochastic simulations of pavement situations where traffic wander and material variability were taken into account (cf. Appendix A) have shown that, in order to satisfy fatigue requirements of typical asphalt mixes, certain minimum support layer stiffnesses need to be observed. Recommended average base stiffnesses for the various traffic classes are shown in Tables 2.2 and 2.3.

**Table 2.2 Minimum Recommended Support Stiffness and Working Strain Ranges for Thin (< 60 mm) Asphalt Surfacing**

<table>
<thead>
<tr>
<th>Traffic Class</th>
<th>Minimum Required Average Base Stiffness (MPa)</th>
<th>Estimated 50th Percentile Working Strain Limit (microstrain)*</th>
<th>Estimated 90th Percentile Working Strain Limit (microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>200</td>
<td>260</td>
<td>520</td>
</tr>
<tr>
<td>Medium</td>
<td>300</td>
<td>160</td>
<td>350</td>
</tr>
<tr>
<td>Heavy</td>
<td>400</td>
<td>100</td>
<td>250</td>
</tr>
<tr>
<td>Very Heavy</td>
<td>500</td>
<td>70</td>
<td>180</td>
</tr>
</tbody>
</table>

* i.e. 50 per cent of all strains are expected to be below the tabulated value.

**Table 2.3 Minimum Recommended Support Stiffnesses for Asphalt Bases**

<table>
<thead>
<tr>
<th>Traffic Class</th>
<th>Minimum Required Average Subbase Stiffness (MPa)</th>
<th>Estimated 50th Percentile Working Strain Limit (microstrain)</th>
<th>Estimated 90th Percentile Working Strain Limit (microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>Asphalt Bases are generally not used in this application</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>Asphalt Bases are generally not used in this application</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Heavy</td>
<td>300</td>
<td>140</td>
<td>220</td>
</tr>
<tr>
<td>Very Heavy</td>
<td>500</td>
<td>100</td>
<td>160</td>
</tr>
</tbody>
</table>

The base stiffness and estimated strain ranges shown in Tables 2.2 and 2.3 are based on the assumption of a static load condition and may therefore be seen as conservative as far as strain estimates are concerned. (For a more detailed description of the derivation of these values, users should refer to note 2 in Section A.2.). The decrease in strain values that is brought about by higher traffic speeds may be offset to some extent by the influence of loading rate on fatigue resistance.

Laboratory measurements have shown that fatigue cracks tend to develop faster when mixes are loaded at higher frequencies. The net effect of increased speed on fatigue resistance is therefore somewhat uncertain, although it is likely that fatigue calculations based on static loading conditions will be conservative if vehicle speeds are greater than approximately 40 km/h.
Although the stiffness of the immediate support layer is the most important element as far as pavement considerations are concerned, this property is in turn determined by many other aspects of the layer itself (e.g. moisture, material quality) as well as by those of the layer below it (i.e. the quality and stiffness of the subbase). These factors are often easier to quantify than stiffness and can provide a reasonable estimate of the support stiffness. The relationship between material quality and stiffness is discussed in the following sections.

Considerations for Rehabilitation

As far as mix design is concerned, the pavement considerations to be taken into account during a rehabilitation project are in many ways easier to assess, but also have a higher degree of variability and are accompanied by more related difficulties than new pavement structures. This is because the existing support conditions can be measured to a certain degree but are influenced by cracking, moisture conditions at the time of testing etc. Standard laboratory test indicators can be used to provide a reasonable estimate of the support stiffness. For projects in which a high design reliability is required (typically heavy to very heavy traffic classes) a relatively precise estimate of the base material can be obtained by using falling weight deflectometer (FWD) measurements. Backcalculation of stiffness using FWD data requires a skilled analyst as well as accurate information on layer thicknesses. A more robust and reliable, albeit less precise indicator of support stiffness is the Base Layer Index (BLI), which is simply the difference between the deflection measured at an offset of 0 mm and that measured at an offset of 300 mm from the FWD load plate.

The condition of the existing surfacing can also provide an indication of the condition and stiffness of the existing base layer or of the support which will be provided by the whole pavement to the overlay. A large percentage area of crocodile cracks and longitudinal cracks in the wheel path indicate that the existing pavement may not provide adequate support for the overlay under prevailing traffic conditions.

Table 2.4 provides a qualitative indication of the relationship between stiffness and various test parameters for unbound base materials. It will be noted that the parameters listed in Table 2.4 are those which are normally measured during trial hole and FWD investigations and should therefore be available for the majority of rehabilitation projects.

It should be noted that the relationship between the stiffness of the material and the individual parameters listed in Table 2.4 can often be erratic. It is therefore important not to rely on any single indicator, but to consider - in a holistic manner - as many as possible of the listed parameters per uniform subsection when existing support conditions are being evaluated.

In the case of bound base materials, stiffness can be evaluated by means of coring and laboratory testing. Some of the laboratory tests that can be used to directly evaluate the stiffness of bound materials are described in Chapter 8. In addition to stiffness measurements, the condition of the bound base needs to be evaluated. Bound base materials which exhibit closely spaced crack patterns may indicate that the lower pavement support is not sufficient and that the cracked layer itself will provide a much lower effective stiffness to the asphalt overlay than may be suggested by measurements on uncracked positions. In these cases structural rehabilitation and/or the application of a highly flexible asphalt surfacing may be required.
Table 2.4  Factors Affecting Stiffnesses of Unbound Support Layers

<table>
<thead>
<tr>
<th>Test Parameter/Condition</th>
<th>Typical Ranges of Values for Unbound Base Materials‡</th>
<th>Relationship to Base Stiffness*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low Quality (&lt;200 MPa)</td>
<td>Medium Quality (200 - 400 MPa)</td>
</tr>
<tr>
<td>California Bearing Capacity Ratio (%)**</td>
<td>60 to 80</td>
<td>&gt; 80</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td>&gt;8</td>
<td>5 to 8</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>&lt; 10</td>
<td>&lt; 6</td>
</tr>
<tr>
<td>Approximate Dynamic Cone Penetrometer Penetration Rate (mm/blow)</td>
<td>&gt;4</td>
<td>2 to 4</td>
</tr>
<tr>
<td>Aggregate Crushing Value (%)</td>
<td>&gt;30</td>
<td>26 to 30</td>
</tr>
<tr>
<td>Base Layer Index†</td>
<td>&gt; 330</td>
<td>150 to 330</td>
</tr>
</tbody>
</table>

*  Direct relationship indicates that the stiffness increases with an increase in the value of the test parameter.
  Indirect relationship indicates that the stiffness decreases with an increase in the value of the test parameter.

**  At in-situ density.
†  FWD Deflection bowl parameter. Values shown are valid for a 40 kN load (plate pressure of approximately 550 kPa).
‡  Material types as defined in TRH14.

Considerations for New Pavement Structures
In the case of new pavement structures, the evaluation of the adequacy of support stiffness has to rely on the design specifications for the pavement and materials. The designer should realize that the success of the asphalt design relies to some extent on the design assumptions for the support and should ensure that these assumptions are validated during construction.

Guidelines for the selection of base materials for different traffic classes can be found in TRH4 and TRH14. Table 2.5 provides a brief summary of suitable base types for different traffic classes.

Considerations for Airports and Industrial Areas
Although many of the design considerations for road pavement structures, airport pavements or industrial areas (e.g. harbours, loading zones) are the same, there are some basic differences between the design objectives for these different types of structures. As a general rule, it may be stated that, in the case of normal road pavement structures, the emphasis is on toughness (i.e. fatigue and repetitive application of comparatively light loads is the major design consideration). In the case of airport pavement structures or asphalt designs for industrial areas, the emphasis shifts somewhat toward strength (i.e. the design considers fewer applications of heavier loads).
Table 2.5 Summary of Suitable Base Types and Associated In-Place Stiffnesses (New Construction)

<table>
<thead>
<tr>
<th>Traffic Class</th>
<th>Suitable Base Types</th>
<th>Typical Stiffness (MPa)</th>
<th>Typical Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Heavy</td>
<td>High Quality Crushed Stone (G1)*</td>
<td>&gt;450</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>Continuously Graded Asphalt Base</td>
<td>&gt;1000**</td>
<td>120 to 180</td>
</tr>
<tr>
<td></td>
<td>Waterbound Macadam</td>
<td>&gt;450</td>
<td>150</td>
</tr>
<tr>
<td>Heavy</td>
<td>Crushed Stone (G1 and G2)</td>
<td>&gt;400</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>Continuously Graded Asphalt Base</td>
<td>&gt;800</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Waterbound Macadam</td>
<td>&gt;450</td>
<td>125 to 150</td>
</tr>
<tr>
<td>Medium</td>
<td>Crushed Stone (G2 and G3)</td>
<td>&gt;350</td>
<td>125 to 150</td>
</tr>
<tr>
<td></td>
<td>Continuously Graded Asphalt Base</td>
<td>&gt;1000**</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Stabilized Base (C3)</td>
<td>&gt;500</td>
<td>150</td>
</tr>
<tr>
<td>Light</td>
<td>Crushed Stone (G2 and G3)</td>
<td>&gt;350</td>
<td>125 to 150</td>
</tr>
<tr>
<td></td>
<td>Natural Gravel (G4 and G5)</td>
<td>&gt;250</td>
<td>100 to 150</td>
</tr>
<tr>
<td></td>
<td>Waterbound Macadam (WM2)</td>
<td>&gt;400</td>
<td>100 to 125</td>
</tr>
<tr>
<td></td>
<td>Stabilized Base (C3 and C4)</td>
<td>&gt;300</td>
<td>100 to 150</td>
</tr>
</tbody>
</table>

* Subbase Requirements will differ depending on climate (see TRH4).

** Asphalt base stiffness may vary considerably depending on not only temperature and vehicle speed, but also on aggregate packing characteristics and binder type and content

Despite this qualification, the pavement considerations that need to be taken into account during the mix design stage are essentially the same as those for road structures, and the recommended base stiffness values shown in Tables 2.2 and 2.3 can therefore be used as a guideline for evaluating support conditions. It should be noted, however, that adequate support essentially protects the asphalt layer from fatigue and shear resulting from excessive flexure. For airport designs, more emphasis should be placed on mix strength and stability, and special tests should be considered to ensure that the mix has adequate resistance to rutting.

### 2.5 Climate

The selection of a mix type, as well as the rating of design objectives, are influenced in three ways by climatic conditions:

**Maximum Temperature**

Temperature is perhaps the most important factor influencing rutting performance. Climatic conditions in which high asphalt temperatures are likely to prevail for large percentage of time require special attention to be paid to rutting resistance. Consideration of the maximum temperature may influence the selection of gradation (including maximum aggregate size), aggregate type and quality, as well as binder type. Figure 2.1 delineates areas with different relative risks of rutting susceptibility based on temperature.
Minimum Temperature
Low temperature is an important factor influencing asphalt fatigue. Climatic conditions in which low asphalt temperatures are likely to prevail for a large percentage of time require special attention to be paid to fatigue resistance. This consideration may also influence the selection of mix type and quality as well as binder type. Figure 2.2 delineates areas with different relative risks of fatigue cracking based on the number of hours in the year when the temperature is below 5°C.

Range of Expected Temperatures
Situations where extreme temperature fluctuations occur during the year increase the demand for a balanced, optimized asphalt mix which has good resistance to rutting at high temperatures, as well as increased resistance to ageing and fatigue at low temperatures. This consideration may especially influence the selection of a binder type.

Rainfall
Mixes located in high rainfall areas or in areas with a large number of rainy days have an increased potential for stripping and may require special attention to be paid to durability issues. Such mixes may also have greater waterproofing requirements, depending on the underlying layers and therefore permeability may become an important issue. Rainfall considerations may thus influence the choice of aggregate type, filler type as well as binder type. Figure 2.3 shows a delineation of different rainfall zones for southern Africa.

Water Sensitivity of Underlaying Layers
In many instances, the thin asphalt surfacing not only protects underlaying layers and carries the tyre loads, but also provides a moisture barrier preventing moisture ingress into the (often) moisture-sensitive pavement layers. Therefore, the layer should be designed to be impermeable if the underlaying layers are sensitive to moisture ingress. Similarly, the asphalt layer and the pavement must have a high degree of impermeability if the materials used in the asphalt itself are susceptible to moisture damage (e.g. stripping).

2.6 HMA Layer Thickness
The thickness of the HMA is often determined by pavement design considerations. This is especially true for thicker layers (e.g. asphalt bases) where the asphalt layer contributes to the overall structural capacity. However, for asphalt less than 50 mm thick, the asphalt layer does not significantly contribute to the structural capacity of the pavement and designers have some leeway in deciding which layer thickness to specify.

The minimum layer thicknesses are generally determined by the maximum stone size. Current specifications limit the ratio of layer thickness to maximum aggregate size to not less than two. Designers should be aware that the risk of aggregate crushing is increased for designs in which the layer thickness to maximum aggregate size ratio is close to this specified limit. Such mixes may also tend to segregate during construction. In situations where the specified compaction may be more difficult to achieve, designers should consider increasing the specified ratio during the design stage. Designers should also be aware that the in-situ properties (e.g. permeability density and voids) of layers in which the layer thickness to maximum aggregate size ratio is greater than 4 may differ significantly from those of the design.
Figure 2.1 Maximum Temperature Zones for South Africa

Figure 2.2 Minimum Temperature Zones for South Africa
Hence, it is recommended that the minimum thickness of an asphalt layer should be 2 to 3 times
the nominal maximum aggregate size, depending on the type of grading. The minimum ratio
between nominal maximum aggregate size and layer thickness for sand-skeleton mixes
manufactured with unmodified binders should be greater than 1:2, whereas that for other mix
types (i.e. stone-skeleton mixes and mixes manufactured with modified binders) should be greater
than 2:5.

The actual minimum layer thickness is that which can be demonstrated to be laid in a single lift
and compacted to the required uniformity and evenness.

In recent years, the industry has seen increased use of asphalt layers with thicknesses of between
20 mm and 30 mm. In South Africa, ultra-thin layers are defined as those with a thickness of 25
mm and less. These layers have been used with success in Europe and are exclusively used to
obtain desired functional properties (particularly riding quality, noise reduction and skid
resistance). While ultra-thin layers may be desirable in some applications, designers should be
aware of several problems that relate to the construction of these layers:

Thinner layers cool more rapidly than thicker layers and therefore allow a shorter time
window in which compaction has to be achieved. Compaction and permeability
specifications are therefore harder to achieve with very thin layers. In cold weather this

Figure 2.3 Rainfall Zones in southern Africa

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Thinner layers cool more rapidly than thicker layers and therefore allow a shorter time
window in which compaction has to be achieved. Compaction and permeability
specifications are therefore harder to achieve with very thin layers. In cold weather this
issue may become critical. It is recommended that the interim recommendations produced by Sabita be consulted (Sabita Manual 224, which deals with hot-mix paving in adverse weather conditions).

At present, there is some difficulty in determining and interpreting volumetric and performance properties of cores obtained from very thin asphalt layers. It is imperative that the base layer be of a high quality with a high quality finish and a proper tack coat. Although the material cost may be reduced by the use of thinner layers, significant savings in overall paving costs are not always achieved.

### 2.7 Other Considerations

**Special Functional Requirements**

Special functional requirements may include:
- High level of noise reduction in urban areas, and
- High skid resistance for high speed applications and in high rainfall areas.

In addition to the selection of mix type, these considerations may have an impact on construction cost and availability of materials.

**Special Geometric Conditions**

Situations where braking, acceleration and turning of heavy vehicles are likely to occur on a regular basis require increased resistance to rutting, skidding and ravelling. Owing to their low resistance to shearing, open-graded mixes should be avoided in these circumstances.

Experience has also shown that there is some difficulty involved in achieving the specified tolerances for support layers at intersections. These factors may complicate the construction (especially the compaction) of the HMA at intersections.

Designers should also be aware that compaction and good surface qualities are difficult to achieve on steep hills and high crossfalls. Such situations require greater attention to be paid to base preparation and finish (including tack coat operations). Application of a modified tack coat should also be considered for such situations.

**Presence of Cracks In Existing Surface**

It was stated earlier (see Section 2.4) that the presence of crocodile cracking and longitudinal cracking may be indicative of a support layer with a low stiffness. Apart from this consideration, the presence of cracks in the existing surfacing may also have a significant impact on the crack initiation time of any asphalt overlay.

Studies have shown that cracks in an existing, older asphalt layer will propagate at a rate of between 25 and 50 mm per year through a newer layer placed over it. Simply reducing the stiffness of the cracked layer for the purposes of mechanistic analysis is not sufficient. Cracks in an existing layer will generate excessively high crack tip stresses in the overlying interface, leading to rapid crack propagation through the overlying asphalt layer. It is therefore imperative that areas with existing cracks be removed and patched before placement of an overlay. The use of a stress absorbing material interlayer (SAMI) or a highly flexible overlay such as a bitumen rubber asphalt may also be considered.
Designers should be aware that the stress conditions which exist in overlays placed over cracked layers cannot be analysed with conventional pavement analysis tools such as those which use multilayer elastic theory, and require special modelling techniques to estimate stresses and strains under load.

Material Availability
Certain mixes are more forgiving than others to variations in aggregate quality and gradation. The various characteristics required from the mix components are described more fully in Chapter 3. At this stage it should be noted that marginal or variable aggregates should not be used in mixes that are highly dependent on aggregate interlock, such as Stone Mastic Asphalt (SMA). Furthermore, if aggregates are unlikely to provide sufficient deformation resistance owing to their quality and variability, a binder of higher viscosity or a modified binder should be selected so as to reduce the potential for segregation and to increase the stability of the mix.

Moisture Damage (e.g. Stripping)
This is a phenomenon where the binder loses its adhesion with the aggregate in the presence of water and hence affects the durability of the mix. The factors which affect stripping are:

- **Aggregate factors**
  - surface texture - rough surfaces improve stripping resistance
  - aggregate type - acidic aggregates are more prone to stripping

- **Binder factors**
  - viscosity - higher viscosities improve stripping resistance
  - chemical composition - some binders are less prone to stripping than others

- **Environment**
  - higher temperatures increase propensity to stripping (i.e. water-vapour interaction with the binder-filler mastic and large aggregate interfaces)
  - higher rainfall increases water supply

- **Mix characteristics**
  - permeability - higher permeability increases the probability of moisture ingress
  - variability - increased variability may result in areas of higher permeability

- **Construction**
  - compaction - areas of poorer compaction will have greater degrees of permeability
  - joints - poor joints will provide avenues for moisture ingress

- **Traffic**
  - high traffic volumes early in the life of the mix will assist in post-construction compaction and hence contribute to the reduction in permeability
  - high traffic volumes may also increase the stripping potential by the development of hydrostatic pressures and hydraulic action

The designer should be aware of these issues and may need to use anti-stripping additives in critical areas.

Considerations Related to Design and Project Specifications
Economic issues that may impact on the mix and materials specifications should be considered at an early stage of the design process. This applies especially to availability of materials. High quality aggregates of a good consistency are often not available or are expensive to transport to certain regions. It is imperative that designers evaluate the availability of aggregates of the
specified quality before project specifications are finalised. Such an evaluation at an early stage may alert the client and tenderer to additional costs that may be incurred through transport or special manufacturing processes needed to produce the desired quality of aggregate.

Situations in which the standard specifications are modified to suit the needs of the project require special attention to be paid to the availability and properties of local materials. Designers should alert tenderers to non-standard project specifications that may have an impact on material availability. This applies specifically to situations in which locally available materials do not meet the project specifications.

2.8 Worksheet for Design Considerations

Table 2.6 is a worksheet for facilitating the selection of a mix type and the rating of design objectives. Use of the worksheet simply involves answering all the questions listed and adding up the scores or ratings for the different design objectives. These ratings are calculated as follows:

i) For each of the questions listed in Table 2.6, do the following: If the answer to the question is “Yes”, circle all the numbers in the same row as the question. If the answer is “No”, do not circle anything and proceed to the next question.

ii) After all the questions have been answered, proceed down the different columns and add all the circled numbers. If the total for a column is greater than three, change this to three. If the total is less than one, change it to one. Write this total down next to the design objective listed at the head of each column.

iii) The totals obtained for the various design objectives (e.g. rutting resistance, fatigue etc.) provide a ranking which can be used to select the required level of performance testing (i.e. more sophisticated and expensive tests are warranted if a rating of 3 is obtained). This ranking can also be used in conjunction with Table 2.7 to select the most appropriate mix type for the application.

The rated design objectives are also used to determine the required performance tests. This issue is discussed in Chapter 4.

Table 2.6 is intended to assist designers in the quantification and consideration of the issues discussed in the preceding subsections. It should, however, be noted that the questions listed in Table 2.6 do not represent all the considerations that may apply to a specific site. Designers should ensure that all possible issues which may relate to traffic, climate, geometry, constructibility and support conditions and which apply to a specific site, are taken into consideration.
<table>
<thead>
<tr>
<th>Item / Issue</th>
<th>Consideration</th>
<th>Influence of Design Objectives on Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic</td>
<td>Is the number of heavy vehicles less than 60 vehicles/lane/day?</td>
<td>-2</td>
</tr>
<tr>
<td>Traffic</td>
<td>Is the number of heavy vehicles less than 200 vehicles/lane/day?</td>
<td>-2</td>
</tr>
<tr>
<td>Traffic</td>
<td>Is the number of heavy vehicles greater than 200 vehicles/lane/day?</td>
<td>1</td>
</tr>
<tr>
<td>Traffic</td>
<td>Is the number of heavy vehicles greater than 600 vehicles/lane/day?</td>
<td>2  2  2  1  1</td>
</tr>
<tr>
<td>Traffic</td>
<td>Are the heavy vehicle speeds often below 50 km/h?</td>
<td>1</td>
</tr>
<tr>
<td>Traffic</td>
<td>Is the section in an urban or noise sensitive area?</td>
<td>2</td>
</tr>
<tr>
<td>Traffic</td>
<td>Is the expected light traffic speed above 80 km/h?</td>
<td>1  2</td>
</tr>
<tr>
<td>Climate</td>
<td>Is the road situated in a high temperature zone (see Figure 2.1)?</td>
<td>2</td>
</tr>
<tr>
<td>Climate</td>
<td>Is the road situated in a high rainfall zone (see Figure 2.3)?</td>
<td>1  1</td>
</tr>
<tr>
<td>Geometry</td>
<td>Does the section include climbing lanes?</td>
<td>1</td>
</tr>
<tr>
<td>Geometry</td>
<td>Does the section include stopping and turning areas?</td>
<td>1  1  2</td>
</tr>
<tr>
<td>Construction</td>
<td>Will construction be conventional, with an implemented QA system?</td>
<td>-1</td>
</tr>
<tr>
<td>Construction</td>
<td>Will construction involve new or labour-intensive methods?</td>
<td>2</td>
</tr>
<tr>
<td>Construction</td>
<td>Is the asphalt thickness less than 40 mm?</td>
<td>1</td>
</tr>
<tr>
<td>Support</td>
<td>Is base stiffness below the recommended value (Table 2.2)?</td>
<td>2</td>
</tr>
<tr>
<td>Support</td>
<td>Is base stiffness more than 30% greater than the recommended value (Table 2.2)?</td>
<td>-2</td>
</tr>
<tr>
<td>Support</td>
<td>Is the base sensitive to moisture ingress?</td>
<td>1  2</td>
</tr>
</tbody>
</table>
2.9 Preliminary Selection of Mix Type and Components

The characterization of a mix type depends primarily on the spatial composition of the mix (e.g. nominal aggregate size, gradation, aggregate, filler and binder characteristics and contents, and the packing characteristics of the mineral components). The selection of a mix type can be optimized by considering the relative demand for each of the different design objectives (i.e. stability, durability, etc.), as determined by the factors discussed in the preceding sections (i.e. expected traffic, pavement and climatic situation, as well as other special design considerations).

Spatial composition
The spatial composition of the mix (i.e. stone- or sand-skeleton) and hence the type of gradation are perhaps the most important choices to be made as far as mix type selection is concerned. The aggregate packing characteristics to a large extent determine the binder content and volumetric properties of the final mix. These elements in turn determine the relative resistance of the mix to deformation, deterioration caused by the environment, etc. Table 2.7 lists the types of gradation covered by these guidelines and also shows a relative rating of the most important performance properties associated with each type.

Nominal Aggregate Size
The selection of a nominal aggregate size is limited by the asphalt layer thickness. Current specifications limit the maximum aggregate size to not more than half of the thickness of the compacted asphalt layer. Designers should however, consider decreasing this limit (i.e. increasing the ratio of layer thickness to maximum aggregate size) whenever conditions are anticipated in which compactibility or segregation may pose problems during construction.

Increasing the nominal aggregate size generally increases the stability of the asphalt but reduces the workability of the mix. Segregation also becomes more problematic when larger aggregate sizes are used. Table 2.8 presents some guidelines for the selection of the maximum aggregate size, which is defined as the smallest sieve size through which 100 per cent of the aggregate passes.
<table>
<thead>
<tr>
<th>Application</th>
<th>Type of Gradation and Binder</th>
<th>Typical Applications</th>
<th>Performance Rating* for</th>
<th>Ease of Design</th>
<th>Rutting Resistance</th>
<th>Durability / Fatigue Resistance</th>
<th>Skid Resistance</th>
<th>Impermeability to Water</th>
<th>Noise Reduction</th>
<th>Ease of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Thin Layer Asphalt</td>
<td>Continuous with 60/70 pen bitumen</td>
<td>surfacing/overlay</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Continuous with bitumen rubber</td>
<td>flexible surfacing/overlay</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Continuous with SBS</td>
<td>flexible surfacing/overlay</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Continuous with SBR</td>
<td>flexible surfacing/overlay</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Continuous with EVA</td>
<td>rut-resistant surfaced</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>SMA with 60/70 pen bitumen</td>
<td>rut-resistant surfaced</td>
<td>3</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>SMA with modified bitumen</td>
<td>rut-resistant surfaced</td>
<td>3</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Open Graded with 60/70 pen bitumen</td>
<td>functional layer</td>
<td>3</td>
<td>4</td>
<td>2</td>
<td>4</td>
<td>N/A**</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Open graded with modified binder</td>
<td>functional layer</td>
<td>3</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>N/A**</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Semi-Gap with 60/70 pen bitumen</td>
<td>flexible surfacing/overlay</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>4†</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Semi-Gap with modified binder</td>
<td>flexible surfacing/overlay</td>
<td>4</td>
<td>2</td>
<td>4</td>
<td>4†</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Semi-Open with bitumen rubber</td>
<td>flexible surfacing/overlay</td>
<td>3</td>
<td>5</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Gap-graded with 60/70 pen bitumen</td>
<td>flexible surfacing/overlay</td>
<td>4</td>
<td>2</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Ultra-Thin Asphalt</td>
<td>SMA with 60/70 pen bitumen</td>
<td>functional layer</td>
<td>4</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>4**</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SMA with modified binder</td>
<td>functional layer</td>
<td>4</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>4**</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Open Graded with 60/70 pen bitumen</td>
<td>functional layer</td>
<td>4</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>N/A**</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Open graded with modified binder</td>
<td>functional layer</td>
<td>4</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>N/A**</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Thick Asphalt Bases</td>
<td>Continuous with 60/70 pen bitumen</td>
<td>base course</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>N/A</td>
<td>3</td>
<td>N/A</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Continuous with 40/50 pen bitumen</td>
<td>base course</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>N/A</td>
<td>3</td>
<td>N/A</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

*1 = Poor, 5 = Good; **Impermeable support layer or membrane required † With rolled-in chips
Table 2.8 Guidelines for the Selection of the Maximum Aggregate Size (sand-skeleton mixes)

<table>
<thead>
<tr>
<th>Maximum Aggregate Size (mm)</th>
<th>Minimum Layer Thickness (mm)</th>
<th>Uses</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5</td>
<td>20</td>
<td>Ultra-thin and thin surfacings</td>
</tr>
<tr>
<td>13.2</td>
<td>30</td>
<td>Thin surfacings</td>
</tr>
<tr>
<td>19.0</td>
<td>40</td>
<td>Conventional Surfacing</td>
</tr>
<tr>
<td>26.5*</td>
<td>60</td>
<td>Thick surfacings and bases</td>
</tr>
<tr>
<td>37.5</td>
<td>80</td>
<td>Asphalt Bases</td>
</tr>
</tbody>
</table>

*Note: In the Western Cape, 26.5 mm BTB has been successfully compacted in a 40 mm layer.

Apart from the maximum aggregate size, three distinct fractions are defined for the purposes of characterizing a specific gradation. These are:

i) Course aggregate: particles retained on the 4.75 mm sieve;
ii) Fine aggregate: particles finer than 4.75 mm, and
iii) Mineral filler; particles passing the 0.075 mm sieve.

Specifications and tests related to the quality of these fractions are discussed in Chapter 3.

Binder Selection
Table 2.9 contains general guidelines for the selection of binders for different mix types and design situations. More detailed information on the characteristics, testing and evaluation of different binder types (including modified binders) is given in Chapter 3.

Modified binders offer advantages in design situations which require mixes with high rutting resistance or above-average flexibility and durability. This is likely to be the case in applications where traffic volumes are high and where the increased cost of using a modified binder is justified. Designers should, however, ensure that the increased cost is justified by the perceived advantage of using a modified binder and attempts should be made to demonstrate such advantages during the design stage by using the performance tests described in Chapter 6, 7 and 8.

Designers should also be aware that the use of modified binders may require special consideration as far as manufacturing, storage and handling are concerned. Under some conditions, mixes manufactured with modified binders are also difficult to place and compact and may require special tests to be performed during construction.
### Table 2.9 General Guide for the Selection of Binder Type

<table>
<thead>
<tr>
<th>Binder Type</th>
<th>Uses and Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>40/50 pen bitumen</td>
<td>Mixes for high traffic applications, where increased stiffness is required. Typically not suitable for situations where support conditions are not of a high standard, or cold regions. Generally only used for thick layers and asphalt bases. Standard specifications apply.</td>
</tr>
<tr>
<td>60/70 pen bitumen</td>
<td>Typical for asphalt surfacings with light to medium traffic. Used for typical asphalt applications in most climatic zones. Standard specifications apply.</td>
</tr>
<tr>
<td>80/100 pen bitumen</td>
<td>Mixes for low traffic applications, where decreased stiffness is required. Typically not suitable for thick layers on a stiff support, or hot regions, unless stabilised (e.g. with fibres). Standard specifications apply.</td>
</tr>
<tr>
<td>Modified Binders &amp; special binders</td>
<td>Used for heavy traffic applications or where special mix requirements exist (e.g. highly flexible or rut resistant mixes).</td>
</tr>
</tbody>
</table>
3. COMPONENT SELECTION

3.1 Aggregate

The physical properties of aggregates are affected by the mineralogy of the parent rock, the extent to which the parent rock has been altered by leaching, oxidation etc., as well as by the processes required to produce graded and blended aggregate. The physical properties of aggregate are generally regarded as the most important aspect of aggregate selection.

Hardness and Toughness
Aggregates are subjected to abrasive wear during the various stages of crushing, screening, manufacturing and placement of HMA and, finally, by normal trafficking. To ensure a stable and rut-resistant HMA, it is essential that aggregates retain a harsh texture throughout these operations. Hardness and toughness are important for providing a rut-resistant mix with a good micro-texture (to ensure good low-speed skid resistance). The test methods used to evaluate hardness and toughness are listed in Table 3.1. It should be noted that these tests are only applied to the coarse aggregate fraction (material retained on the 4.75 mm sieve size). Hence, designers and manufacturers should be wary of weaker areas in the quarry which may only manifest in the finer aggregate fractions.

Durability and Soundness
Durability and soundness are a measure of the ability of aggregates to resist breakdown and disintegration under the action of the environment. Environmental forces that tend to degrade aggregates include wetting and drying and freezing and thawing cycles. Durability is to a certain extent both a physical and a chemical property of aggregate. The sulphate soundness test described in Table 3.1, however, primarily evaluates the physical resistance to degradation caused by fracturing along fine cracks and foliation. At present, there is no standard specified test for evaluating the durability of aggregates. One test worth considering, if the aggregate is suspect, is the ethylene glycol soundness test. In this test the ethylene glycol causes potentially deleterious clay minerals within the aggregate particles to swell, thus breaking down the aggregate. This is evaluated visually. The recommended test protocol is described in Appendix A.

Particle Shape and Surface Texture
The stability and workability of HMA is greatly affected by the shape of particles. For heavy and very heavy traffic applications, particles should be angular to ensure good stability. However, the current crushing processes and origin of the rocks may dictate the level of angularity and the designer often has only limited control over aggregate angularity. For lighter traffic applications, more rounded aggregate may be tolerated to promote workability. Aggregates with flat, thin and elongated particles should be avoided as their shape may prevent proper compaction.

The surface texture of aggregates affects the strength and workability of asphalt. Aggregates with a rough, sandpaper-like surface are conducive to high stability mixes and are therefore needed for situations in which high rutting resistance is important (typically heavy to very heavy traffic applications). A smooth surface texture promotes workability but this is accompanied by a drop in stability. Smooth-textured aggregates may therefore be considered for low traffic applications where workability is perhaps more important than stability. Rough-textured aggregates tend to result in higher void contents in the mix. Although smooth-textured aggregates may be easier to
coat with bitumen, the bond between the aggregate and the binder is not as strong and durable as when rough-textured aggregates are used.

The surface texture of aggregates to be used in surfacings is also an important determinant of skid resistance. Harsh, sandpaper-like textures promote greater micro-texture, a property which is related to low-speed skid resistance.

The aggregate polishing value test provides a measure of the durability of the surface texture, or loss of surface texture as a result of polishing. The flakiness index test is a standard test used to evaluate particle shape. The particle index test provides a combined measure of particle shape and surface texture. Evaluation criteria for these tests are defined in Table 3.1.

**Cleanliness**

Cleanliness of aggregates refers to the absence of foreign and deleterious materials in aggregate fractions. Such foreign materials include vegetation, shale, soft particles, clay lumps or clay coatings on aggregate surfaces and excess dust from crushing operations. Cleanliness can be ensured by proper quarrying and storage, as well as by washing of aggregates in exceptional cases. Tests which can be used to quantify aggregate cleanliness are defined in Table 3.1.

**Variability**

The quality of the rock in aggregate quarries often varies, resulting in the crushing process producing material with different fractions and particle shapes. When very narrow gradation envelopes are specified, these may only be consistently achieved at increased cost because special processes are needed. For mixes such as SMAs, which rely on stone-to-stone interlock for deformation resistance, small variations in gradation may affect performance significantly. Potential aggregate producers should be consulted when tight envelopes are specified in project specifications. Generally, it is more beneficial to provide sufficient bins for the different aggregate fractions at the mixing plant, than to expect the aggregate supplier to consistently provide an aggregate blend that conforms to the specified gradation envelope. This also allows for reconstitution of the grading by means of blending, should small adjustments be needed to ensure the required properties. This, however, would require a particular HMA plant to be specified.

**Test Methods for Evaluating Physical Properties of Aggregates**

The test methods used for the evaluation of the physical properties of aggregates, together with evaluation criteria are summarized in Table 3.1. The following should be noted regarding some of the tests listed in Table 3.1:

- **Los Angeles abrasion test:** This is not a standard test in South Africa. The correlation between LA abrasion test results and aggregate performance in service appears to be poor.

- **Sulphate soundness test:** This test has been criticised for not being able to accurately predict the field performance of certain aggregates. The test primarily measures resistance to freezing and thawing and its relevance to Southern African conditions is therefore questionable. The test may, however, be used to obtain a relative evaluation of durability, or to alert the designer to characteristics which require further investigation.
<table>
<thead>
<tr>
<th>Property</th>
<th>Test</th>
<th>Designation</th>
<th>Acceptance Criteria and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardness / Toughness</td>
<td>Fines Aggregate Crushing Test (Dry 10% FACT) (-13.2 +9.5 mm fraction)</td>
<td>TMH1; B1</td>
<td>Minimum of: 160 kN: HMA surfacings and base, excluding open-graded and SMA mixes 210 kN: Open-graded surfacings and SMA</td>
</tr>
<tr>
<td></td>
<td>Aggregate Crushing Value (ACV)</td>
<td>TMH1; B1</td>
<td>Maximum of: 25%: HMA base and surfacings, excl. open-graded and SMA mixes 21%: Open-graded surfacings and SMA</td>
</tr>
<tr>
<td></td>
<td>Los Angeles Abrasion Test</td>
<td>ASTM C131 and ASTM C535</td>
<td>No Standard Specified  Typical values are 10%: Very hard aggregate 60%: Very soft aggregate</td>
</tr>
<tr>
<td>Durability / Soundness</td>
<td>Sulphate Soundness Test</td>
<td>ASTM C88</td>
<td>No Standard Specified  12% to 20% is normally acceptable Some specifications specify no more than 12% loss after 5 cycles</td>
</tr>
<tr>
<td></td>
<td>Methylene Blue Adsorption</td>
<td>Appendix A</td>
<td>No Standards Specified  Indicators: &lt;5: High quality filler  &gt;5: Additional testing required</td>
</tr>
<tr>
<td></td>
<td>Ethylene Glycol</td>
<td>Appendix A</td>
<td>Visual evaluation (no standards specified)</td>
</tr>
<tr>
<td>Particle Shape and Texture</td>
<td>Flakiness Index Test</td>
<td>TMH1; B3T</td>
<td>Maximum values for HMA surfacings  19 mm and 13.2 mm aggregate: 25 (grade 1*) or 30 (grade 2)  9.5 mm and 6.7 mm aggregate: 30 (grade 1*) or 35 (grade 2)  HMA bases: 35 (applies to -26.5 mm/+10.9 mm and -19.0 mm/+13.2 mm sieve fractions)</td>
</tr>
<tr>
<td></td>
<td>Particle Index Test</td>
<td>ASTM D3398</td>
<td>No Standard Specified  Typically, rounded particles have a particle index of 6 to 7; Highly angular, crushed particles have particle indices above 15. A particle shape index of 14 normally separates natural and crushed sands.</td>
</tr>
<tr>
<td></td>
<td>Polished Stone Value Test</td>
<td>SABS 848</td>
<td>Minimum of 50 for continuously graded, open-graded and SMA surfacings  Minimum of 45 for gap-graded surfacings</td>
</tr>
<tr>
<td>Fractured Faces</td>
<td></td>
<td></td>
<td>HMA surfacings: At least 95% of all particles should have at least three fractured faces HMA bases: At least 50% of the plus 4.75 mm fractions should have at least one fractured face</td>
</tr>
<tr>
<td>Absorption</td>
<td>Water Absorption, coarse aggregate</td>
<td>TMH1; B14</td>
<td>Maximum of 1% by mass</td>
</tr>
<tr>
<td></td>
<td>Water Absorption, fine aggregate</td>
<td>TMH1; B15</td>
<td>Maximum of 1.5% by mass</td>
</tr>
</tbody>
</table>
### Table 3.1 Tests used to Evaluate the Physical Properties of Aggregates (continued)

<table>
<thead>
<tr>
<th>Property</th>
<th>Test</th>
<th>Designation</th>
<th>Acceptance Criteria and Comments</th>
</tr>
</thead>
</table>
| Cleanliness  | Sand Equivalent Test              | TMH1\(^*\); B19 | Minimum of:  
- 50: total fines fraction  
- 30: natural sand fraction to be mixed with aggregate (where permitted)  
  
Clay lumps and friable particles  
ASTM C1426  
No standard specified  
The percentage of clay lumps and friable particles is normally limited to 1 per cent |

\(^*\) As defined in TRH14 (currently under revision)

#### Required Chemical Properties: Binder Adhesion

The only chemical property of an aggregate which directly impacts on HMA performance is its affinity to the binder. This property is related to the surface chemistry of the aggregate. The bond that forms when bitumen coats the surface of aggregate can weaken in the presence of water. In the case of aggregates that have a greater affinity for water than for bitumen (termed hydrophilic, or water-loving aggregates), the binder film on the aggregate may become detached, or ‘strip’ in the presence of water. Evidence suggests that hydrophilic aggregates tend to be acidic in nature, while hydrophobic (water-hating) aggregates are basic in nature. Aggregates with rough, slightly porous surfaces that are clean and have been exposed to some degree of environmental ageing generally have better stripping resistance.

Several tests have been proposed to evaluate the susceptibility to stripping of aggregate-binder combinations. None of these tests can consistently identify mixes with high stripping potential. However, the Modified Lottman test (ASTM D4867\(^*\)) is generally regarded as the best test for evaluation of the stripping potential of an aggregate. This test is discussed in Chapter 8.

### 3.2 Aggregate Properties Required for Design Calculations

In addition to the physical and chemical properties listed above, two other aggregate properties need to be evaluated during the design of HMA: (i) relative density, and (ii) absorption. These properties do not influence the suitability of the aggregate for use in HMA, but are required for the volumetric calculations described in Chapter 5.

The relative density of a material is the ratio of the density of a material to that of water at the same temperature. Since the density of water is 1.0 gram per ml at 25 °C, the relative density of aggregate at 25 °C (the standard test temperature) can simply be expressed as:

\[
\text{Relative Density} = \frac{\text{mass}}{\text{volume}} \quad \text{(Eq. 3.1)}
\]

To conceptualize the spatial composition of asphalt mixes, volume considerations are most often used. However, these considerations need to be controlled by means of mass measurements, for which the relative density of the aggregate is needed. To do this, the relationship expressed by Equation 3.1 is used as follows:
Thus the relative density of aggregate is needed to enable volumetric calculations to be made. Quantities can be defined or specified in terms of volume, and then monitored by replacing each volume quantity by the appropriate mass divided by the appropriate relative density.

Aggregate surfaces contain cracks and cavities which may absorb a certain volume of binder and water, or may remain filled with air. Because of this, there are different ways in which the relative density can be measured. These measurements differ in the manner in which the voids and cavities on the aggregate surface are interpreted during density calculations.

Figure 3.1 shows a schematic illustration of an aggregate coated with a film of binder. The aggregate has a cavity which has been partly filled by the binder. Binder which has entered such cavities is referred to as absorbed binder. Figures 3.2 to 3.4 illustrate the differences between the bulk, effective and apparent relative densities. An understanding of the differences between these relative densities is important, since it may affect the accuracy of void calculations that are made in the mix design stage.

Consideration of Figures 3.2 to 3.4 and Equation 3.1 will show that the bulk relative density is always equal to or less than the effective relative density. The apparent relative density is always equal to or greater than the effective relative density. Thus:

$$ \text{Bulk Relative Density} \leq \text{Effective Relative Density} \leq \text{Apparent Relative Density} $$

The bulk relative density to some extent assumes that there are no voids in the aggregate which can be penetrated by the binder. Since this is not actually the case, the effective volume of binder will be less, and consequently the volume of voids between coated particles may be greater than that shown by the design calculations. Thus, if the bulk relative density alone is used for calculation of void contents, the actual void contents may be greater than that shown by the design calculations.
**BULK RELATIVE DENSITY (BRD):**

- Assumes no absorption;
- Measured voids in mix design calculations include voids filled with absorbed binder and voids in aggregate not filled with binder;
- Use of Bulk Relative density in mix design calculations may lead to overestimate of actual voids in mix;

BRD assumes aggregate looks like this:

\[
\text{BRD} = \frac{\text{Mass of Oven Dry Aggregate}}{(\text{Vol. Aggregate}) + (\text{Vol. Voids filled with Binder}) + (\text{Vol. Voids not Filled with Binder})}
\]

Figure 3.2 Definition and Illustration of Bulk Relative Density

**EFFECTIVE RELATIVE DENSITY:**

- Takes absorption into account;
- Falls between Bulk and Apparent Relative Densities;

Effective Relative Density assumes aggregate looks like this:

\[
\text{Effective Relative Density} = \frac{\text{Mass of Oven Dry Aggregate}}{(\text{Vol. Aggregate}) + (\text{Vol. Voids not Filled with Binder})}
\]

Figure 3.3 Definition and Illustration of Effective Relative Density

**APPARENT RELATIVE DENSITY:**

- Voids in aggregate not filled with binder are included in design calculations;
- Use of Apparent Relative density in design calculations may cause underestimate of actual voids in mix;

Apparent Relative density assumes aggregate looks like this:

\[
\text{Apparent Relative Density} = \frac{\text{Mass of Oven Dry Aggregate}}{\text{(Vol. Aggregate)}}
\]

Figure 3.4 Definition and Illustration of Apparent Relative Density
The situation is reversed for apparent relative density. This is because the apparent relative density assumes that all the surface cavities in the aggregate voids will be fully penetrated by the binder. This is not actually the case, since part of these cavities will be too fine or small for the binder to penetrate (although still large enough for water to penetrate). Thus the apparent relative density over-estimates the increase in available void space due to bitumen absorption. Therefore, if the apparent relative density is used in design calculations, the actual voids will be less than that shown by the design calculations.

The most correct aggregate relative density to use in air void calculations would therefore be the effective relative density, although the effective relative density is difficult to determine. When the effective relative density is used in air void calculations, the calculated voids are truly those of the HMA mix. Although the effective relative density does not appear directly in the calculation of the total percentage voids in the compacted mix, it is used to calculate the maximum theoretical relative density of the mix, a quantity which directly appears in void calculations.

The voids in the mineral aggregate (VMA) is another important volumetric quantity which requires the aggregate relative density in its calculations. The VMA is calculated by subtracting the following from the total sample volume: (i) the volume of the aggregate, plus (ii) the volume of voids filled and (iii) not filled with binder. These latter three volumes constitute the total volume which is used to calculate the bulk relative density (see Figure 3.2). Thus the VMA calculations are based on the bulk relative density of the aggregate.

Bitumen absorption in aggregates tends to follow a hyperbolic relationship with time. For this reason it is recommended that the absorption potential of aggregates be determined at different ageing times. If absorption is determined at zero ageing time and at any two other ageing times, the absorption versus time information can be used to predict the ultimate absorption potential of the binder-aggregate combination.

Studies suggest that the most appropriate method for calculating bitumen absorption in aggregates is through Rice’s method for measuring relative density. Measurements are taken after 4 hours ageing at 143°C.

In cases where the absorption of binder is very small, the various relative densities will tend to be very similar. Designers should, however, note that the absorption of aggregates is an important quality which can affect design calculations and which may cause poor performance if not adequately accounted for. Typically, water absorption values for fine aggregates are below 1.5 per cent (by mass), while those for coarse aggregates are below 1 per cent. Designers should be aware that aggregates with absorption capacities which lie above this limit may require special test methods to accurately determine volumetric quantities. Details of the calculation of effective and bulk relative densities of aggregates are provided in Chapter 5.

### 3.3 Filler

Filler is defined as the material passing the 0.075 mm (or 75 μm) sieve. In an asphalt mix the filler generally serves two purposes: (i) it acts as an extender for the binder to stiffen the mastic and the mix, thereby improving stability; and (ii) it acts as a void-filling material and can therefore be used to adjust aggregate gradations and volumetric mix properties. Some fillers are also used to improve the bond between the binder and the aggregate. Specific fillers such as fly-ash
can also be used to improve mix compactability. Table 3.2 shows various types of filler with the most important characteristics of each.

It is important that adequate amounts of filler are available to ensure that the mix has adequate cohesion, providing sufficient internal tensile strength and mix toughness to resist shearing forces. The latter is particularly relevant for sand-skeleton mixes, where mix cohesion is a major contributing factor to the provision of resistance to permanent deformation. Whereas this would be less important to stone-skeleton mixes, as the resistance to permanent deformation is mainly provided by stone-to-stone contact and aggregate interlock, adequate mastic viscosity would still need to be provided to prevent binder run-off to occur during the manufacturing, transport and placement of such mixes.

While the filler may serve the purposes mentioned above, the presence of too much or of too “active” a filler may cause the viscosity of the hot mastic during the mixing and compaction process to increase to such an extent that adequate compaction is not possible in the field. Tests carried out on a range of South African aggregates (the minus 0.075 fraction crusher dust was used) have shown that the binder-with-filler may stiffen dramatically beyond a certain filler-binder ratio. Figure 3.5 indicates that, at a temperature of 60°C, the viscosity ratio (defined as the filler-binder viscosity divided by the binder viscosity) increases significantly when the filler-binder ratio exceeds 1:2. Figure 3.6 indicates that, at a temperature of 135°C, the viscosity ratio increases significantly when the filler-binder ratio exceeds 1:1. These two temperatures (i.e. 60°C and 135°C) reflect the upper limits of road performance temperature and an average field compaction temperature, respectively. In the Western Cape, it is recommended that the filler-binder ratio of wearing course mixes should not exceed 1.5, particularly for thin-layer mixes that cool more rapidly during paving and compaction. Because of their heat retention, higher filler-binder ratios can be allowed in thick asphalt bases (i.e. a ratio of approximately 1.6).

The effect of the viscosity of the mastic at high filler-binder ratios should not affect the compactibility of SMA mixes, for which the compaction and stability characteristics are determined primarily by stone-to-stone contact.

Table 3.2 Filler Types and Characteristics

<table>
<thead>
<tr>
<th>Filler Type/Origin</th>
<th>Characteristics and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrated Lime (active filler)</td>
<td>Improves adhesion between binder and aggregate;</td>
</tr>
<tr>
<td></td>
<td>Improves mix durability by retarding oxidative hardening of binders;</td>
</tr>
<tr>
<td></td>
<td>Low bulk density and high surface area;</td>
</tr>
<tr>
<td></td>
<td>Relatively high cost;</td>
</tr>
<tr>
<td></td>
<td>Monitor effect on stiffness to ensure compactability;</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>Low bulk density;</td>
</tr>
<tr>
<td></td>
<td>Relatively high cost;</td>
</tr>
<tr>
<td></td>
<td>Variable characteristics require greater control;</td>
</tr>
<tr>
<td>Portland Cement (active filler)</td>
<td>Relatively high cost;</td>
</tr>
<tr>
<td></td>
<td>Monitor effect on stiffness to ensure compactability;</td>
</tr>
<tr>
<td>Baghouse Fines</td>
<td>Variable characteristics require greater control;</td>
</tr>
<tr>
<td></td>
<td>Some source types may affect mix durability;</td>
</tr>
<tr>
<td></td>
<td>Some types may make mixes sensitive to small changes in binder content.</td>
</tr>
</tbody>
</table>
Figure 3.5 Influence of Filler-binder-Ratio on Mastic Viscosity at 60°C

Figure 3.6 Influence of Filler-binder-Ratio on Mastic Viscosity at 135°C
Evaluation of Physical and Chemical Properties of Fillers

Filler content and its quality also have an effect on the durability of a mix. In theory, natural fillers which have an excess of clay minerals or adsorption potential may cause early hardening and stripping of the mix. Baghouse fines, which tend to be variable in nature, may result in the stiffening of the mastic in places, leading to differential compaction densities that in turn may affect the durability of the mix.

The Methylene Blue test provides an indication of the amount and activity of clay minerals in the filler. Experience shows that methylene blue values (MBV) of 5 or less are indicative of high quality filler. Fillers with methylene blue values above 5 should be further evaluated by means of hydrometer analysis and Atterberg limits determinations. It should be noted that there are several methods for performing the Methylene Blue test. The test protocol recommended for use in South Africa is described in Appendix A.

In addition to the physical and chemical characteristics of different types of fillers, designers also have to consider their relative cost, availability and storage potential. Active fillers such as Portland cement and hydrated lime are readily available and of consistent quality, but cost more than waste products such as fly dust, fly ash or baghouse fines. Except for Portland cement, all of these fillers are easy to store and handle. Active fillers generally have a greater effect on binder stiffness than inactive fillers and should be used rationally, particularly when a fair amount of natural rock filler is present in the aggregate.

3.4 Binder Evaluation

Binder Types used in HMA

Penetration Grade Binders

In South Africa, conventional, or ‘straight’ bitumen is classified by means of the penetration (pen) grading system. There are four standard penetration grades: 40-50, 60-70, 80-100 and 150-200 pen. These binders are evaluated according to the SABS 30710 specification. The penetration grade provides a relative indication of the binder viscosity, with higher penetration grades corresponding to lower viscosities. Of the penetration grades named above, only the first three are commonly used for HMA production, the 60-70 pen grade being by far the most commonly used binder type (see Table 2.8).

It should be noted that high-stiffness bases, using a very hard paving grade bitumen (20-30 pen), have become popular in Europe (especially France) for use in heavy-trafficked applications. Their attraction lies in that they provide excellent load spreading ability and are designed to have an ‘indeterminate’ or ‘perpetual’ life.

Modified binders are produced by blending a polymer or natural hydrocarbon with straight bitumen. The modification of bitumen with polymers can improve the performance of binders significantly, but does increase the cost of production and monitoring. Benefits that may be derived from polymer modification include:

- improved consistency and decreased temperature susceptibility;
- improved flexibility, resilience and toughness (increases mix durability);
- improved stability and cohesion (increases resistance to permanent deformation), and
- improved binder-aggregate adhesion.
Polymers which are commonly used for binder modification can be classified as elastomers or plastomers. Elastomers generally make the binder more elastic. Effectively, this means that the binder can stretch more without fracturing. The binder also becomes stiffer as it stretches, thereby offering more resistance to deformation. Plastomers, on the other hand, make the binder more stiff and rigid. This increased stiffness reduces the overall strain, thereby also effectively decreasing the permanent strain. The most commonly used modifiers in South Africa are:

- Rubber crumbs, consisting of recycled natural and synthetic elastomers;
- Styrene-Butadiene-Rubber (SBR) (a synthetic elastomer);
- Styrene-Butadiene-Styrene (SBS) (a synthetic elastomer);
- Ethylene-Vinyl-Acetate (EVA) (a plastomer), and
- Natural hydrocarbons.

The most important properties of these modifiers are summarized in Table 3.3.

**Table 3.3 Characteristics of Commonly used Modifiers**

<table>
<thead>
<tr>
<th>Modifier</th>
<th>Ranking for Relative Increase/Decrease* in</th>
<th>Suggested Applications (Applied for HMA with/where:)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low Temp. Flexibility</td>
<td>High Temp. Stability</td>
</tr>
<tr>
<td>Rubber crumbs</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Styrene-butadiene-styrene (SBS)</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Styrene-butadiene-rubber (SBR)</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Ethylene-Vinyl-Acetate (EVA)</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Natural hydrocarbons</td>
<td>0</td>
<td>3</td>
</tr>
</tbody>
</table>

* Increase or decrease relative to the use of an unmodified binder;  
  1 = Small Increase; 3 = Large Increase;  
  -1 = Small Decrease; -3 = Large Decrease.
The use of polymer-modified binders generally requires some specialization with respect to blending and mixing operations as well as to testing and monitoring of consistency on site. Designers should be aware that the use of modified binders invariably requires more precise and specialized quality control during construction. Some modifiers have greatly reduced storage potential and may also have compatibility problems with some binders. Modified binders should not be specified in situations where limited quality control will be exercised during construction. Problems may also arise when modified binders are specified in contracts which involve inexperienced contractors.

The increased cost of materials and construction that is incurred when modified binders are used should be justified by the perceived increase in HMA performance. Such a perceived increase in performance may rest on the experience of the client or designer, but should preferably be demonstrated during the design process by means of performance testing and by comparison with conventional products.

Binder Properties, Testing and Evaluation

A sound HMA design has three main requirements with respect to the binder: (i) stability (resistance to permanent deformation); (ii) resistance to cracking and (iii) durability or resistance to environmental deterioration. In addition to these performance aspects, tests are also conducted to ensure purity as well as safety upon heating of the binder. Because of the complex nature of the behaviour of binders, no single test can provide a conclusive evaluation of the above requirements. For this reason, several tests are used to evaluate the physical and chemical characteristics of bituminous binders. The properties that are measured with these tests are directly or indirectly related to the stability, fatigue resistance and durability of the binder and can therefore be used to evaluate the suitability of a binder for specific applications.

The durability of a binder is generally evaluated by means of simulated ageing. Binder properties are measured before and after simulated ageing and the change in properties upon ageing is used to evaluate durability. For stability, use is made of the absolute values of several test parameters as obtained before or after simulated ageing.

Two types of simulated ageing are used to evaluate binder durability: (i) short-term simulated ageing by means of the rolling thin film oven test (RTFOT), and (ii) long-term simulated ageing by means of the pressure ageing vessel (PAV). The first of these two modes of ageing is designed to simulate the ageing that takes place during mixing, transport and paving operations. The latter mode of ageing simulates the ageing that takes place during long-term exposure to the environment. For South African conditions and owing to the lack of availability of test apparatus, only RTFOT ageing is generally used. The use of the RTFOT apparatus for simulated long-term ageing is being investigated. While recommendations for simulating long-term ageing in the RTFOT test do not exist as yet, it is hoped that they can be developed reasonably cost-effectively.

For conventional binders, the RTFOT is normally performed at an oven temperature of 163°C. However, experience has shown that this temperature may be too low to allow certain types of modified binders to flow and coat the glass. For these cases, it is proposed that the RTFOT be performed with a steel rod inside the binder container to ensure proper binder spreading.
Other Considerations with Respect to Binder Testing and Evaluation

Table 3.4 shows the standard tests and specifications used for bituminous binders. This table also shows some statistical parameters which define the distribution of observed test values for a number of binders. The typical observed values shown can be used to evaluate the relative performance of a binder which meets the standard specification. Care should be taken, however, not to interpret these values as specifications in themselves. For low traffic applications the standard specifications will offer an acceptable evaluation of binder performance. For HMA designs for high traffic applications, designers can use the range of typical values to obtain further assurance of expected binder performance and to determine whether there is a need for more comprehensive performance testing.

The tests outlined above pertain to the performance of binder as part of hot-mix asphalt. These tests are considered to be the most relevant for determination of expected binder performance. Another test which relates to purity is the Spot Test, performed according to AASHTO T10211.

Designers should be aware that the physical properties of binders may change during the course of an HMA contract. In particular, there may be large differences between binder properties measured at the design stage and those of binder which has been heated in bulk storage over a period of time. Binder testing and evaluation should therefore be conducted as part of the normal quality control process during construction. Such testing will ensure quality control as well as confirm that the properties of the constructed mix correspond with those of the laboratory design mix.

While such tests do not have to include all of the abovementioned tests for a given design level, testing of basic properties such as penetration and softening point before and after RTFOT ageing is recommended.
<table>
<thead>
<tr>
<th>Property</th>
<th>Standard Specification (SABS 307-1972)</th>
<th>Typical Values for Some Properties (60/70 pen binders only)</th>
<th>Comments/Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration at 25°C, (dmm)</td>
<td>40-50  60/70 pen.  80-100</td>
<td>Average 15th percentile 85th percentile</td>
<td>Indication of stability (higher values may indicate greater susceptibility to rutting of the mix)</td>
</tr>
<tr>
<td>Softening Point (R&amp;B) (°C)</td>
<td>49-59  46-56  42-51</td>
<td>- - -</td>
<td>Indication of temperature at which binder becomes more plastic</td>
</tr>
<tr>
<td>Viscosity at 60°C (Pa.s)</td>
<td>220-400  120-250  75-150</td>
<td>- - -</td>
<td>Indication of stability (lower values may indicate greater susceptibility to rutting of the mix)</td>
</tr>
<tr>
<td>Performance after RTFOT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Viscosity at 60°C (Pa.s)</td>
<td>N/S*  N/S  N/S</td>
<td>301  208  378</td>
<td>Values close to or lower than 15th percentile value indicate that the binder may be susceptible to rutting</td>
</tr>
<tr>
<td>Change in Viscosity at 60°C</td>
<td>300 (max)</td>
<td>205  166  239</td>
<td>Values close to or higher than 85th percentile may indicate low durability of the binder</td>
</tr>
<tr>
<td>Softening Point (R&amp;B) (°C)</td>
<td>52 (min)  48 (min)  44 (min)</td>
<td>53  50  55</td>
<td>Indication of the temperature at which binder becomes more plastic</td>
</tr>
<tr>
<td>Increase in softening point</td>
<td>9 (max)</td>
<td>4  3  5</td>
<td>Values close to or higher than 85th percentile may indicate low durability of the binder</td>
</tr>
<tr>
<td>Retained Penetration (% of original)</td>
<td>60 (min)  55 (min)  50 (min)</td>
<td>- - -</td>
<td>Indication of stability (higher values may indicate greater susceptibility to rutting of the mix)</td>
</tr>
</tbody>
</table>

* N/S = No specification
4. **VOLUMETRIC DESIGN AND PERFORMANCE TESTING**

**General**
After the design objectives have been determined, the mix type has been selected and the various components have been evaluated, the actual design process can begin. The basic design procedure consists of the following steps:

- **Sample Preparation** (including sourcing of suitable materials and the proper sampling thereof), **Compaction and Volumetric Calculations**. This involves compacting specimens at different binder contents and understanding both the compaction characteristics as well as the volumetric and engineering characteristics of the mix as determined in the laboratory and as expected immediately after construction and during its lifetime in the field.

- **Engineering Properties**. Where there is uncertainty and risk, some engineering properties should be determined to increase the level of confidence to that which is required for the application, and to verify the properties expected from the volumetric calculations.

- **Field Trials**. During construction some field trials should be carried out to assess whether the field mixing and compaction processes can produce a layer with the required properties.

The basic steps required for volumetric design and performance testing are shown in Figure 4.1. Although the overall process as illustrated in Figure 4.1 is common to all mix types, specific criteria and procedures apply to individual mix types. The design processes that apply to specific mix types can be found in the sections describing the design of specific mix types. Some issues pertaining to mix design are discussed below.

**Spatial Considerations**
Before the selection of a target gradation and the calculation of volumetric design parameters is started, designers should be aware of the intended spatial composition of the planned mix. In particular, designers should be aware of the packing characteristics of the planned mix type and how this influences the volumetric design parameters. The type of skeleton structure that is aimed for in the design should be kept in mind and the evaluation and selection of the gradation should ensure that the appropriate packing mechanism is attained. This concept is shown graphically in Figure 4.2. As explained in Appendix B, two opposing packing mechanisms govern the packing of aggregates:

- **Substitution**, in which the space occupied by the fine aggregate fraction is replaced by an increase in the concentration of the coarse aggregate fractions. This mechanism applies to sand skeleton mixes.

- **Filling**, in which the spaces between coarse aggregates are filled by an increase in the concentration of fine aggregate. This mechanism applies to stone skeleton mixes.

These two packing mechanisms serve different purposes and have different advantages and disadvantages as far as stability, durability and compactibility are concerned. The selection of a target gradation and analysis of volumetric parameters should be relevant for the particular type of packing mechanism that is aimed for in the design.
Selection of Target Gradation
The processes described in the sections on the design of specific mix types assume that a single gradation is selected before the binder content selection process is started. This means that the experimental design is limited to the number of binder contents at which the design is performed. In some situations, it may be beneficial to use more than one gradation in the mix design process. For such cases, a proper experimental design should be set up so that the effects of different gradations on the volumetric properties can be assessed in a rational and consistent manner.

Sensitivity analysis using the theoretical models (optional)
Once the target gradation has been selected, designers should evaluate the gradation to determine a target binder content, as well as investigate the effect of variations in the gradation on volumetric properties. This evaluation can be performed using any model (such as COMPACT) which relates volumetric properties to gradation and aggregate characteristics. Any anomalies which may arise should be investigated. For example, if a COMPACT analysis indicates that the volumetric properties may move out of the specified range for small variations in certain aggregate fractions, then the gradation should be adjusted to ensure a more robust design.
Mixing and Conditioning of Samples
The recommended procedure for mixing of the aggregate and the binder is as prescribed for the Marshall method (see TMH15, Appendix to method C2). No sample conditioning is performed before compaction.

Sample Compaction and Volumetric Evaluation
The method proposed in the new South African HMA design procedure differs from the previous Modified Marshall method in that, instead of five different binder contents, three binder contents are used with a single compactive effort. More importantly, the procedure requires that a thorough understanding of the workability and volumetric properties of the mix be obtained for different compactive efforts.

While neither the Marshall compaction process nor the gyratory shear compaction process accurately simulates field compaction, it is essential that an evaluation of likely field density and related properties be made so that performance predictions based on these densities and not only on the density obtained at the end of the laboratory compaction procedure, can be made. To obtain this information a device is fitted to the Marshall compaction hammer which can monitor the specimen height after each blow so that the densification of the specimen can be monitored as in the Gyratory Shear compaction process13.

The HMA design process does not include different compaction levels for different traffic situations, but rather an understanding of the volumetric and related performance properties of the mix as compacted in the laboratory. This understanding should enable the volumetric and
performance properties as they are likely to develop during the compaction process in the field, to be predicted, i.e. immediately after construction to the specified density, and then after trafficking to the midpoint of the life of the layer.

The binder content and minimum field density after construction should then be selected, based on the above understanding and not on arbitrary criteria such as 4 per cent void content in the Marshall Specimen and on field compaction to 97 per cent of the Maximum Theoretical Relative Density (by Rice’s method) less Marshall voids, i.e. 7 per cent void content. In certain mixes such as SMA’s a void content of 7 per cent may be excessive and may result in high permeability, while in others such as continuously graded “dry” mixes on poor bases, the above density may be too difficult to achieve. Similarly, in mixes placed in areas with low traffic it may be possible to reduce the expected field void content, after a few years’ trafficking, to 3 per cent or less instead of the usually assumed 4 per cent. An understanding of the workability of the mix and its situation with regard to compaction may determine the target minimum construction void content.

Designers are encouraged to investigate the effect of aggregate variability in respect of grading and shape on the likely void contents which may occur in the field. After the design binder content has been selected, the aggregate properties and binder content can, for example, be used in a model such as the COMPACT14 computer programme. These quantities can then be varied within the range expected and the results assessed. If the COMPACT14 analysis indicates that the final void content may decrease to below 2 per cent, the binder content may have to be reduced. Alternatively, if the void content immediately after construction is always expected to be greater than 7 per cent, then the binder content may have to be increased.

**Note:** Although the above outlines the approach recommended at this stage, it is important to note that little practical experience is currently available on linking the void contents obtained by the Marshall and gyratory shear compaction processes with the field compaction void contents over the lifetime of the layer. Therefore, it is proposed that the above process be implemented as an interim procedure for a period during which the necessary experience should be built up to enable the procedure to be finalized.

**Performance Testing**

The number and types of performance tests required for a specific application are determined by the rating of design objectives, as determined by the worksheet illustrated in Chapter 2 (Table 2.9). This rating is dependent on several factors, including traffic, climate, pavement structure and geometry.

**Note:** It is important for designers to understand that no clear qualitative relationships between the different performance tests and actual field performance have been derived to date. Therefore, the evaluation of some performance test results is based on recommended ranges of test values associated with different situations, rather than on a fixed criterion.

Designers therefore have to be familiar with the typical ranges of test results that are obtained from the various performance tests. The typical ranges of test values allow designers to have some freedom to assess the suitability of the mix for a given situation and also to assess the risk associated with a specific mix or design situation.
For most design situations, the evaluation of rutting and fatigue poses the greatest challenge as far as mix performance evaluation is concerned. Because of this, the procedures for rutting and fatigue evaluation are discussed in some detail in Chapters 6 and 7, respectively. These chapters focus on the test and analysis process, as well as on the interpretation of results.

In addition to rutting and fatigue, mix durability and permeability also need to be evaluated. Permeability is assessed by means of the test procedure described in Chapter 8. Mix durability (resistance to stripping) is assessed by means of the Marshall Immersion Index and the Modified Lottman test described in Section 8.3. It should be noted that the durability of a mix is strongly influenced by the properties of the binder and that, therefore, a proper evaluation of binder properties as discussed in Section 3.4 should be performed.
4.1 Densely Graded Sand-Skeleton Mixes

General Mix Description
Densely graded sand-skeleton mixes are the most commonly used mix type for low to high traffic situations. These mixes derive their name from their aggregate packaging characteristics, which is designed to attain high density and dense packing of aggregates. In a densely graded mix the spaces between the coarse aggregate particles are filled with the well-graded portions of finer aggregate. For this reason, densely graded mixes derive their stability from a sand skeleton (unlike stone mastic asphalt and open-graded mixes, which rely on a stone skeleton). In the context of these design guidelines, densely graded sand-skeleton mixes refer to continuously graded, semi-gap-graded and gap-graded mixes.

Well-designed continuously graded mixes offer a reasonable balance between stability and durability. However, several checks and balances have to be considered to ensure that a proper balance is attained between these two properties and that the mix will neither be too difficult to compact nor be prone to rutting.

Component Selection
The selection of aggregate, filler and binder is described in Chapter 3. All these components need to be evaluated and, if necessary, re-selected before the mix design procedure described below is started.

Selection of a Design Gradation
Typical gradation envelopes for densely graded mixes with various nominal maximum stone sizes are shown in Appendix C. In the selection of the nominal maximum stone size, the considerations with respect to layer thickness need to be taken into account (see Chapter 2, Table 2.7). It should also be noted that coarser mixes will generally have greater stability but may also exhibit high permeability immediately after construction, particularly if target densities are not met in isolated areas.

Binder Content Selection and Evaluation of Compactibility
The selection of an optimum binder content for densely graded mixes does not rely on a fixed process with rigid volumetric design criteria. Rather, the procedure requires a thorough understanding of the compaction and volumetric characteristics of the mix at different binder contents and for different compactive efforts. In order to allow designers to assess the mix behaviour during compaction and traffic densification, the Modified Marshall device was developed\(^{13}\). This device allows the mix density to be monitored with increasing number of blows.

The process for the design of densely graded mixes requires that the designer should balance and evaluate several mix properties at the same time. The factors to be taken into account include:

Traffic:
Traffic affects the ultimate degree of compaction that the mix will undergo in the field after it is constructed. It should therefore be taken into account when estimating the approximate laboratory compactive effort to be used when assessing the properties of the mix at its likely ultimate voids content.
Compactibility:
Mixes which compact easily will rapidly approach an ultimate density and those which are less workable will gradually densify with compactive effort. It is important to understand the degree of likely ultimate densification which has been effected in the laboratory relative to field and traffic compaction. The modified Marshall device, like the gyratory shear compactor, allows the compaction characteristics of the mix to be evaluated. This allows designers to evaluate the range of likely densities which will be achieved during the lifetime of the mix and to predict whether problems with compaction or inadequate voids are likely to be experienced in the field.

Initial Voids Content after Construction:
In the past the specified maximum void content after construction was typically set at 97 per cent of Maximum Theoretical Relative Density (MTRD) (determined by Rice's method) minus the design void content determined in the Marshall Design Procedure. The design void content was generally set at 4 per cent and hence the specified maximum construction void content was set at 7 per cent. In many cases, particularly for continuously-graded mixes, this resulted in permeable mixes being placed on lightly trafficked roads, which did not compact any further owing to the light traffic, resulting in mixes which oxidised rapidly over time. On heavily trafficked roads severe deformation occurred in places owing to excessive densification under traffic, resulting in inadequate ultimate void contents and lack of deformation resistance.

In the new design method ranges of initial and final void content criteria are proposed, depending on the traffic expected to be encountered. In the derivation of these criteria, variability was also taken into account to ensure that absolute minimum limits for void content are met at isolated points where the actual void content may differ from the design void content.

Final Void Content after Trafficking:
The minimum void content after traffic compaction should never be less than 2 per cent, otherwise binder expansion resulting from increased road temperatures as well as variations in the aggregate gradation and binder contents may result in isolated sections with no voids, in which case the aggregate will float in the binder and all resistance to deformation will depend solely on the properties of the binder/filler system.

In the new design method it is proposed that the expected final void content after trafficking be set and that a further check on the ultimate likely minimum void content also be made to avoid sudden dramatic deformation when very high temperatures are coupled with heavy traffic loads.

Laboratory Compaction and Construction and Traffic Compaction:
Neither the Marshall nor the gyratory shear compactor simulates field compaction accurately. It would appear that 75 Marshall blows on one side of the laboratory specimen provides a rough estimate of typical construction compactive effort for most densely graded mixes. In the case of gyratory shear compaction, 50 gyrations at an angle of 1.25º (600 kPa pressure, 30 gyrations per minute) appears to simulate construction densities for some binder contents and mixes. However, for the same mix with lower binder contents, 50 gyrations may overestimate construction densities, while at higher binder contents, the same number of gyrations may underestimate construction densities. Therefore, at this stage, use of the gyratory shear compactor is only proposed for mixes which are to be placed under heavy traffic, as it provides a better indication of the likely ultimate density that may be achieved than the Marshall compaction device.
Table 4.1 shows the criteria for the selection of an optimum binder content. These criteria ensure that the permeability and density requirements after construction are met and, at the same time, that the stability requirements based on minimum void content are met after trafficking.

**Table 4.1:** Guidelines on Voids Criteria for Densely Graded Mixes to Select an Optimum Binder Content

<table>
<thead>
<tr>
<th>Traffic Level*</th>
<th>Allowable Void Content Range after 75 Marshall blows† (to simulate field compaction)</th>
<th>Allowable Void Content Range after additional compaction to simulate trafficking</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>Light</td>
<td>3.5%</td>
<td>5.5%</td>
</tr>
<tr>
<td>Medium</td>
<td>4.5%</td>
<td>6.5%</td>
</tr>
<tr>
<td>Heavy</td>
<td>5.5%</td>
<td>7.5%</td>
</tr>
</tbody>
</table>

Min. voids content of 1.5% after 300 gyrations with Gyratory compactor, according to SHRP testing protocol
Permeability of the mix within acceptable norms (cf. Section 8.8)

Very Heavy      | 6.0%    | 8.0%    | 75 + 75            | 4.5%    | 5.5%    |                        |
Min. voids content of 2.5% after 300 gyrations with Gyratory compactor, according to SHRP testing protocol
Permeability of the mix within acceptable norms (cf. Section 8.8)

* See Table 2.1
† Here, 75 blows is the total number of blows per face, as applied in the Modified Marshall device

The criteria shown in Table 4.1 present a window through which the Marshall compaction curve should pass. For very heavy traffic, an additional assurance of stability is provided by the specification of a minimum void content after 300 gyrations with a Gyratory compactor, tested according to the Strategic Highway Research Programme (SHRP) protocol (angle of 1.25°, 600 kPa pressure and 30 gyrations per minute). Also, the designer needs to ensure that the permeability of the mix, tested in accordance with the method described in Section 8.8, is acceptable.

Figures 4.3 to 4.5 show the voids criteria checkpoints after compaction and simulated trafficking, plotted together with Marshall compaction curves at three binder contents, for a medium continuously graded mix. The curves shown in these figures are the averages of three replicates compacted at each binder content. During an actual project, designers should first plot the three replicates at each binder content to ensure that no one replicate deviates excessively from the other two.

The optimum binder content should be selected such that the compaction curve at the optimum binder content passes between the checkpoints for field compaction and simulated trafficking. The manner in which the optimum binder content is selected for different traffic conditions can best be explained by means of the examples shown in Figures 4.3 to 4.5.
Light Traffic Conditions (Figure 4.3)

The compaction curves for all three binder contents pass above the voids criteria checkpoints. This indicates that under light traffic conditions - the mix may exhibit high permeability and a high rate of oxidation. Thus the compaction data indicate that, for light traffic, a higher binder content, (say) 6.3 per cent, can be selected. Three replicates should be obtained at this binder content. If the average of the three passes through the checkpoints, the optimum binder content can be finalised at 6.3 per cent.

Medium Traffic Conditions (Figure 4.4)

The compaction curve for the 6.0 per cent binder content passes through the voids checkpoints and suggests that a binder content of 6.0 per cent may be appropriate for medium traffic conditions. However, the final voids content after 120 blows is very close to the minimum allowable void content after simulated trafficking. Thus a slightly lower binder content, (say) 5.8 per cent, can be considered. Again, three replicates should be obtained at this binder content to ensure that the average of the three passes through the checkpoints.

Heavy Traffic Conditions (Figure 4.5)

Figure 4.5 indicates that a binder content of 5.5 per cent will ensure that the final void content after simulated traffic (150 blows for heavy traffic conditions) will be at the upper limit of the allowable final void content. This figure also clearly shows that a binder content of 6.0 per cent is too high for the expected traffic conditions and will lead to a mix with insufficient stability. For a binder content of 5.5 per cent, the void content after 75 blows suggests that the mix may be difficult to compact. A slightly higher binder content, (say) 5.7 per cent, could therefore be considered as the optimum binder content. Should there be limited experience with the mix and/or the environment in which it will be applied, increased confidence can be obtained by subjecting the mix (prepared at optimum binder content) to gyratory testing. After having been subjected to 300 gyrations, the voids content of the mix should be greater than 1.5 per cent.

Very Heavy Traffic Conditions (Figure 4.6)

Figure 4.6 indicates that a binder content of 5.5 per cent will ensure that the final void content after simulated traffic (150 blows for heavy traffic conditions) will be within the limits of the allowable final void content. Also, a binder content of 5.5 per cent will ensure that the void content after 75 blows will be within the allowable void content range of 6.0 and 8.0 per cent voids. To verify the design, it is recommended that the mix prepared at optimum binder content be subjected to 300 gyrations, yielding a voids content in excess of 2.5 per cent.

It is recommended that, in addition to the guidelines offered in Table 4.1, the Marshall compaction voids be plotted against the natural logarithm of the number of Marshall blows, and that the slope of the regression curve be determined. If the slope of the regression curve is steeper than 5 per cent, this could be indicative of mixes with refusal void contents (i.e. after the mix having been subjected to 300 gyrations with the Gyratory compactor) of less than 2 per cent and, hence, be susceptible to permanent deformation. Hence, if the slope of the regression curve is steeper than 5 per cent, Gyratory tests should be performed.

The design process described in the preceding paragraphs is intended to ensure that the spatial and volumetric parameters are within the appropriate ranges for different traffic conditions. In many cases, the traffic considerations will also dictate the level of performance testing that is required to validate the volumetric design. However, in some cases (for example when a road has low traffic volumes, but is situated in a high temperature zone where there are steep slopes and
slow moving traffic) a higher level of performance requirements may be appropriate. This applies specifically to rutting resistance.

Chapter 2 provides guidelines for the rating of different design objectives. Procedures for validating the volumetric mix design through performance testing are described in Chapters 6 to 8.

![Figure 4.3 Example of Optimum Binder Content Selection for Light Traffic Conditions](image)

![Figure 4.4 Example of Optimum Binder Content Selection for Medium Traffic Conditions](image)
Figure 4.5 Example of Optimum Binder Content Selection for Heavy Traffic Conditions

Figure 4.6 Example of Optimum Binder Content Selection for Very Heavy Traffic Conditions
4.2 Stone Mastic Asphalt (after: NCHRP Report 9-8/4\(^{16}\))

**General Mix Description**
Stone mastic asphalt (SMA) is an excellent mix type for use as a surfacing under heavy traffic conditions. The primary characteristic of a properly designed SMA is its good resistance to permanent deformation. SMA mixes also have a relatively high durability and generally also better wet-weather skid resistance and noise reduction characteristics than densely graded mixes.

SMA mixes rely on a stone skeleton to provide stability. This skeleton is provided by a coarse, gap-graded aggregate structure, which is filled with a mastic consisting of binder, filler and fibres. A key to the successful design of SMA mixes is the selection of a proper gradation, coupled with the selection of a correct mastic content to ensure that stone-to-stone contact is maintained. By comparison with densely graded mixes, SMA has a relatively high binder and filler content. Fibres are normally added to the mix to prevent drain-down of the binder.

**Design Considerations**
SMA mixes are best utilized as thin surfacings on heavily trafficked roads and at intersections. SMA mixes require high quality aggregate and a consistent gradation and binder content to maintain stability throughout the life of the mix. It is thus not economical to use SMA mixes in a structural layer. In practice, this means that the thickness of an SMA layer is generally limited to 40 mm or less.

**Design Procedure**
The design method described below is based primarily on the work of Brown et al., as described in NCHRP Report 9-8/4\(^{16}\). The method is based on volumetric considerations, the criteria for voids, VMA, etc being derived from experience and volumetric principles. SMA mixes are normally relatively easy to compact. Therefore, to avoid crushing of the aggregate, fewer gyrations or Marshall blows are used for SMA mixes than for densely graded mixes.

**Component Selection**
The component selection process as outlined in Chapter 3 should be followed prior to finalization of a gradation or the selection of a design binder content. In addition to the procedures outlined in chapter 3, the following specific considerations apply to the selection of components for SMA mixes:

**Filler and Fibres**
Fibres are normally added to SMA mixes to stabilize the mastic to prevent drain-down during construction. Cellulose fibres or mineral fibres can be used. Typical contents of these two material types are 0.3 per cent and 0.5 per cent by mass of the mix, respectively.

**Selection of a Design Gradation**
Typical gradations for SMA mixes with various nominal maximum stone sizes are shown in Appendix C. In the selection of a nominal maximum stone size the considerations with respect to layer thickness should be taken into account (see Chapter 2, Table 2.7).
DETERMINE VCAdrc FOR SELECTED GRADATION (AASHTO T19)

DETERMINE MIXING TEMPERATURE

SELECT A TRIAL BINDER CONTENT (see Note 1)

MIX MATERIAL AT TRIAL BINDER CONTENT AND DETERMINE MTRD (See Note 2)

COMPACT SPECIMENS USING 50 BLOWS PER FACE (Marshall) OR 100 GYRATIONS (Gyratory)

CALCULATE THE VMA, VCAmix AND VOIDS

DETERMINE TWO ADDITIONAL BINDER CONTENTS (see Note 4)

PLOT VOIDS, VMA AND VCAmix VERSUS BINDER CONTENT

USE PLOTS TO SELECT OPTIMUM BINDER CONTENT (See Note 5)

PERFORM DRAINDOWN TEST AT OPTIMUM BINDER CONTENT

ARE BASIC CRITERIA MET? (See Note3)

YES

START

ESTABLISH TRIAL GRADATION

NO

REPEAT PROCESS

Figure 4.7 Process for the Selection of Optimum Binder Content for SMA Mixes16
The successful performance of SMA mixes are highly dependent on the particle composition and spatial arrangement of particles. More specifically, it depends on whether or not stone-to-stone contact is achieved and maintained under load. Specific steps should therefore be taken to ensure that the stone skeleton of the SMA mix is not overfilled with mastic. To evaluate whether this is the case, the voids in the coarse aggregate (VCA) of the compacted mix have to be less than the VCA of the coarse aggregate without mastic. The latter quantity is determined by means of the dry rodded VCA test (AASHTO T197), and is termed VCA_{dry}. The VCA of the compacted mix is termed VCA_{mix} and is calculated as follows:

\[ VCA_{mix} = 100 - \left( \frac{G_{mb}}{G_{CA}} \cdot P_{CA} \right) \]  

(Eq. 4.1)

Where: 
- \( G_{mb} \) = Bulk relative density of the compacted mixture;
- \( G_{CA} \) = Bulk relative density of the coarse aggregate fraction, and
- \( P_{CA} \) = Percentage of coarse aggregate in the total mixture.

**Binder Content Selection**

The process for the selection of an optimum binder content for SMA mixes is shown in Figure 4.6. The following notes apply to this figure:

**Note 1:** For coarse aggregate with a bulk relative density of 2.75 or greater, a trial binder content of 5.5 per cent is recommended. For aggregates with a bulk relative density of less than 2.75 a trial binder content of 6.0 per cent is recommended.

**Note 2:** Sufficient material should be mixed to allow 4 samples to be compacted. One sample is used for determination of the MTRD of the mix. The other three are compacted and serve as replicates for determination of void content, VMA and VCA_{mix}.

**Note 3:** For the trial binder content, the following criteria apply
- Minimum VMA of 17.0 per cent;
- VCA_{mix} should be less than VCA_{dry}, and
- Minimum void content of 3.0 per cent.

If any of these conditions are not met, the gradation has to be adjusted (see Table 4.2 for suggested remedial action) and the process for binder content selection has to be started again.

**Note 4:** If the void content for the trial binder content is close to or below 4.0 per cent, the two additional binder contents should be less than the initial trial binder content. If the voids content for the trial binder content is above 4.0 per cent, one of the additional binder contents should be above and the other below the initial trial binder content.

**Note 5:** The optimum binder content should be that at which the mix best meets the criteria shown in Table 4.2.
Table 4.2 Volumetric Design Criteria for SMA mixes

<table>
<thead>
<tr>
<th>Property</th>
<th>Criterion*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air Void Content (%)</td>
<td>4.0**</td>
</tr>
<tr>
<td>minimum</td>
<td></td>
</tr>
<tr>
<td>VMA (%)</td>
<td>17.0</td>
</tr>
<tr>
<td>VCA_{mix} (%)</td>
<td>less than VCA_{dc}</td>
</tr>
</tbody>
</table>

* Criteria shown apply to Marshall and Superpave Gyratory compacted specimens.
** For a design objective rating of less than 3 for rutting, the minimum void content can be less than 4.0 per cent, but should not be less than 3.0 per cent.

Performance Testing
The required performance tests are determined on the basis of rated design objectives. A process for the rating of design objectives is provided in Chapter 2. Guidelines for the evaluation of rutting and fatigue performance are provided in Chapters 6 and 7, respectively. For almost all design situations in which SMA is used, moisture susceptibility and tensile strength tests would be the minimum test requirements. Since SMA is normally used in high traffic situations which require a high-stability mix, wheel tracking tests would normally also be performed.

Because SMA’s are relatively easy to compact, a construction density which is somewhat higher (say, 1-2 per cent higher) than those selected for densely graded mixes can be specified.

Addressing of Mix Deficiencies
SMA mixes which fail to meet the volumetric design criteria, exhibit poor rutting performance, durability or excessive drain-down, need to be redesigned. Table 4.3 provides guidelines for correcting deficiencies in SMA mixes. It should be noted that most of the volumetric calculations depend on the accuracy of the measurements and calculations associated with the bulk relative density (BRD) and other parameters. Incorrect measurement or calculation of these quantities may jeopardize the entire design process. A first step to any corrective action in the design process should therefore be the verification of the accuracy of the volumetric design calculations.
Table 4.3 Problems and Potential Solutions for SMA mixes (based on the information provided in NCHRP Report 9-8/4)

<table>
<thead>
<tr>
<th>Problem/Deficiency</th>
<th>Possible Cause</th>
<th>Potential Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>VMA too low</td>
<td>Percentage passing 4.75 or 0.075 too high</td>
<td>Reduce the percentage passing the 4.75 and/or 0.075 mm sieves</td>
</tr>
<tr>
<td></td>
<td>Excessive aggregate breakdown</td>
<td></td>
</tr>
<tr>
<td>VMA too high</td>
<td>Percentage passing 4.75 and/or 0.075 too low</td>
<td>Increase the percentage passing the 4.75 and/or 0.075 mm sieves</td>
</tr>
<tr>
<td>Voids too low</td>
<td>VMA too Low</td>
<td>Reduce the binder content or increase the VMA</td>
</tr>
<tr>
<td></td>
<td>Binder content too high</td>
<td></td>
</tr>
<tr>
<td>Voids too high</td>
<td>VMA too high</td>
<td>Increase the binder content of reduce the VMA</td>
</tr>
<tr>
<td></td>
<td>Binder content too low</td>
<td></td>
</tr>
<tr>
<td>VCA too high</td>
<td>Percentage passing the 4.75 mm sieve too high</td>
<td>Reduce the percentage passing the 4.75 mm sieve</td>
</tr>
<tr>
<td>Draindown too high</td>
<td>Insufficient filler content</td>
<td>Increase stabilizer content</td>
</tr>
<tr>
<td></td>
<td>Insufficient stabilizer</td>
<td>Change the type of stabilizer</td>
</tr>
<tr>
<td></td>
<td>Proportion of coarse aggregate too high</td>
<td>Modify the gradation to reduce the percentage of coarse aggregate</td>
</tr>
</tbody>
</table>
4.3. Open-graded Mixes (after: Sabita Manual 1718)

General Mix Description
Open-graded asphalt (also called porous asphalt) is used primarily as a surfacing layer to improve skid resistance and visibility in wet weather and also to reduce noise pollution. The void content for open-graded mixes is typically in the order of 20 per cent, with most voids being interconnected. Open-graded mixes rely on a stone skeleton for stability, and generally have good resistance to permanent deformation, provided that a high quality aggregate is used.

Because of their high permeability, open-graded mixes have to be underlain by an impervious layer. Fatigue cracking which, in the case of other asphalt mixes, may result in ingress of water into the pavement structure, is thus not a serious design consideration in the case of open-graded mixes. Severe fatigue cracking or brittleness may, however, lead to ravelling, therefore the abrasion resistance of open-graded mixes is specifically tested during the design stage.

The stiffness of open-graded mixes is generally significantly less (typically 50 per cent) than that of more densely graded mixes. Open-graded mixes are therefore not well suited for use as a structural layer (such as a base course) and are best used only as thin surfacings. Because of their open void structure, open-graded mixes are more prone to suffer from durability problems related to the environment. Stripping potential and ravelling are therefore serious design considerations in the case of open-graded mixes. To maximize durability, the binder content should be as high as possible for the given voids content and stability requirement.

Because of their open structure and the need for a higher than usual binder content, open-graded mixes are prone to exhibit binder drain-down during construction. The amount of binder drain-down that may take place therefore also needs to be taken into consideration during the mix design stage.

Design Considerations
For open-graded surfacings that are selected mainly for wet weather safety reasons, a thickness of 40 mm is recommended. Thinner layers may tend to clog up more rapidly, with a resulting drop in wet weather safety performance. If noise attenuation is the most important design objective, it is recommended that a thickness greater than 40 mm be used.

Because of their porous structure, open-graded mixes need to be underlain by an impervious layer such as densely graded asphalt. If the underlying layer consists of cement treated material, it is recommended that a stress-absorbing membrane interlayer (SAMI) be used to retard reflection cracking and to assist in keeping water out of the pavement structure.

Special considerations are also needed to facilitate drainage of the water collected in the open-graded asphalt layer. The design of the open-graded mix should therefore make provision for the water to pass through the porous layer to lateral collecting drains or onto the shoulder. An adequate crossfall is also needed to prevent water from being trapped in the open-graded asphalt layer.
A lateral transition zone or cut-off drain should be constructed between an open-graded asphalt layer and other sections consisting of impermeable material. This is needed to prevent sheet flow of water entering from the open-graded asphalt layer onto the impervious layer, thereby impairing road safety.

Where not to use Open-graded Asphalt
In order to derive maximum skid resistance and noise reduction from open-graded mixes, it is important that open-graded mixes remain relatively free of dust, pollutants or other deleterious material. Open-graded mixes have also been found to be less resistant to the shearing action imposed by stopping and turning wheels than other types of mixes and tend to ravel where such traffic conditions are found. For these reasons, and also to ensure that the pavement structure is protected from ingress of surface water, it is recommended that open-graded mixes not be used in the following situations:

At intersections;
In industrial areas where there is extensive wear from abrasion, spillage of fuels or any other contamination from deleterious material, which may tend to clog up the void structure of the mix;
In areas with permeable or soft support layers, and
On roads or in areas which are frequently soiled by waste or windblown dust and sand.

Guidelines for Component Selection
The first step in the design procedure is the component selection. Apart from the general guidelines for component selection that are given in Chapter 3, the following apply specifically to open-graded mixes:

Aggregates
It should be noted that specific guidelines are provided for the evaluation of aggregates to be used for open-graded mixes. Since open-graded mixes derive their stability from the stone skeleton, it is vital to ensure that hard and durable aggregates are selected.

Binders
Designers should strive to ensure that the binders used for open-graded mixes have a low temperature and loading rate susceptibility. These properties can be evaluated by means of the dynamic shear rheometer. Binders should also have high durability, as indicated by the change in softening point and viscosity during RTFOT ageing. In view of the high void content of open-graded mixes, the binder should also have a relatively high viscosity. This requirement is generally in opposition to the durability requirement, and for this reason modified binders should be considered for open-graded mixes.

Fillers
A mineral filler content of between one and two per cent is recommended to enhance the adhesion properties of the binder. Fibres can be added to increase binder film thickness and to reduce stripping potential and binder drain-down. Owing to the increased viscosity of the mastic, open-graded mixes with fillers are also more rut resistant than other types of mix.
Guidelines for Selection of the Gradation
The selection of a gradation for open-graded mixes depends on binder type, and also on whether fibres are used. Nominal maximum stone sizes vary between 9.5 and 13 mm. In the selection of a nominal maximum stone size the considerations with respect to layer thickness should be taken into account (see Chapter 2, Table 2.7). Typical gradations for open-graded mixes with various nominal maximum stone sizes are given in Appendix C. The porosity of open-graded mixes increases as the gap size increases (i.e. as the distribution of the coarse aggregate fractions is narrowed).

Table 4.4: Guidelines for Gradation and Binder Content Selection for Open-graded Mixes

<table>
<thead>
<tr>
<th>Mastic/Binder Type</th>
<th>Typical Binder Content (% by mass of mix)</th>
<th>% Passing 2.36 mm sieve</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional</td>
<td>4.2 to 4.8</td>
<td>&lt; 15</td>
<td>Gap in gradation is situated between 2.36 and 9.5 mm</td>
</tr>
<tr>
<td>Conventional plus fibres</td>
<td>4.5 to 5.5</td>
<td>13 to 15</td>
<td>Gap in gradation is between 2.36 and 6.7 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Filler content is typically 5 per cent for a fibre content of 0.3 to 0.5% (by total mass of mix)</td>
</tr>
<tr>
<td>Polymer modified binders</td>
<td>4.5 to 5.6</td>
<td>10 to 18</td>
<td>Polymer modified binders with fibres can also be used</td>
</tr>
<tr>
<td>Bitumen rubber</td>
<td>5.5 to 6.5</td>
<td>11</td>
<td>3 to 4% filler is typically used (including ± 1% lime)</td>
</tr>
</tbody>
</table>

Binder Content Selection

A detailed description of the selection of an optimum binder content can be found in Sabita Manual 17. The general process for the selection of an optimum binder content for is illustrated in Figure 4.7. The following notes apply to Figure 4.7:

Note 1: The quantity of material that is mixed at each binder content should be sufficient to allow 5 Marshall briquettes to be compacted, as well as for the Schellenberg draindown test and MTRD determination. For conventional binders, it is recommended that the binder content range used in the design should start from 3.5 per cent. For modified binders or mixes containing fibres, it is recommended that the binder content range should start at 4.0 per cent.
START

DETERMINE MIXING TEMPERATURE (see Section 4)

ESTABLISH TRIAL GRADATION

DETERMINE MIXING TEMPERATURE (see Section 4)

MIX MATERIAL AT DIFFERENT BINDER CONTENTS AND CONDITION (see Note 1)

UNCOMPACTED MATERIAL

COMPACT BRIQUETTES USING 50 BLOWS PER SIDE (Marshall hammer)

MEASURE MTRD

MEASURE BRD AND DETERMINE VOIDS

PERFORM SCHELLENBERG DRAINDOWN TEST

PERFORM CANTABRO ABRASION TEST

PLOT VOIDS AND ABRASION RESISTANCE VERSUS BINDER CONTENT

DETERMINE OPTIMUM BINDER CONTENT USING LIMITING CRITERIA FOR VOIDS, ABRASION, DURABILITY AND BINDER RUN-OFF (see Note 2 and Figure 4.9)

Figure 4.8 Process for the Selection of Optimum Binder Content for Open-Graded Mixes
The optimum binder content is determined on the basis of a number of limiting criteria which determine the maxima and minima within which the optimum binder content should fall. These parameters are: voids content, abrasion resistance, durability and binder run-off. To ensure good durability, a minimum binder content of 4.5 per cent is normally specified. Figure 4.8 provides an example of the selection of an optimum binder content. In this figure, a minimum void content of 20 per cent has been selected for the design. This provides the first maximum below which the design binder content should fall (maximum 1). The binder run-off provides another limiting parameter (maximum 2). For this example, however, maximum 1 (voids criteria) is lower than maximum 2 (run-off criteria). Thus maximum 1 will determine the maximum allowable binder content. The abrasion loss determines the minimum allowable binder content.

The design binder content is specified as the average of the higher of the minimum binder contents (determined by abrasion resistance and durability criteria) and the lower of the maximum binder contents (determined by void content and binder run-off criteria). Thus, in Figure 4.8, the design binder content will be determined by taking the average of minimum 1 and maximum 1.

Figure 4.9 Determination of Optimum Binder Content for Open-Graded mixes
**Performance Testing**

The performance tests are selected on the basis of the rated design objectives. For high to very high traffic levels, it is recommended that performance testing be conducted on open-graded mixes to evaluate durability and moisture susceptibility. Durability testing for open-graded mixes is a specialised procedure which differs from that used for other mix types and is described below.

**Durability**

In addition to the specification of a minimum allowable binder content (typically 4.5 per cent), durability is also evaluated using aged specimens in the Cantabro test procedure. The procedure is as follows:

Condition and compact 5 briquettes at the design binder content. Age the specimens according to the procedure specified for the Cantabro test with ageing (see Section 8.5). The results should be evaluated to ensure that no individual test result exhibits more than 50 per cent abrasion loss and that the average abrasion loss for the 5 specimens does not exceed 30 per cent.

**Moisture Susceptibility**

Moisture susceptibility is evaluated using the Modified Lottman procedure (AASHTO T283), as described in Section 8.3.
4.4 Large Aggregate Mixes for Bases (After: Sabita Manual 13\textsuperscript{19})

**General Mix Description**
LAMBS are used primarily for asphalt bases as the structural support layer in heavy duty pavements. Heavy duty pavements are those expected to carry traffic volumes in excess of 10 million E80s during their design period. The runways of high-volume airports and certain loading facilities could fall into this category. LAMBS obtain their strength and resistance to permanent deformation primarily from aggregate interlock. This is readily achieved by using large top size aggregates such as 37.5 mm and 53 mm. Because LAMBS are used in the base layer of a pavement, factors such as skid-resistance, ravelling and noise generation do not have to be considered.

By definition, LAMBS do not presuppose a specific grading. Typically, LAMBS are designed with continuous aggregate gradations, although any gradations that promote stone-on-stone contact are acceptable. Open and gap-graded mixes are not considered as LAMBS. Open-graded mixes are too permeable and gap-graded mixes do not provide aggregate interlock. Generally, very densely graded LAMBS (designed such that the aggregate gradation fall on the maximum density line) should be avoided. The use of large aggregates in an asphalt mix, results in a considerable decrease in the surface area and VMA of the mix with the consequence that lower binder contents may be used.

**Design Considerations**
Consideration must be given to structural and environmental aspects which must be taken into account prior to the mix design stage. Other considerations include layer thickness.

When LAMBS are used as a structural support layer in a heavy-duty pavement, consideration should be given to factors influencing the response of the LAMBS mixture under extreme loading conditions. The design of the LAMBS mix must therefore be such as to provide stiffness and enhance the resistance to permanent deformation of the pavement structure. Under extreme loading conditions, the refusal density of a LAMBS mix must be such that there are sufficient voids in the mix (more than 2 per cent) even after years in the field. For this reason, consideration must be given to stable gradations. Furthermore, the design binder contents of LAMBS are chosen to be on the dry side of the VMA curve after standard Hugo or gyratory shear compaction.

The voids in the mix of LAMBS after construction will influence the permeability of the mix. In cases where adequate drainage or impermeable wearing courses are used, the void content after construction may be between 6 and 8 percent, otherwise the void content after construction should preferably be below 6 per cent. The use of natural sand and of binder contents slightly higher than normal have been found to improve the workability and compactibility of LAMBS mixes when permeability of the mix is undesirable, although the benefits should be weighed against the loss in mix stability under heavy loading.

Segregation of the aggregate in LAMBS during manufacture and construction is often a problem caused by poor mix gradation. Segregation should be limited, if not totally eliminated, as it allows ingress of water, which could lead to stripping.

In the design of LAMBS consideration should be given to fatigue. These mixes are often designed with low binder contents and their position in a pavement structure is such that the

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underside of the LAMBS layer is subjected to tensile stresses, which may lead to cracking and eventually fatigue. The stiffness of the subbase layer should therefore also be taken into consideration. Stiffness of LAMBS is an important design consideration, particularly because of the load-spreading ability of these layers to protect the underlying layers and the subgrade. The interaction between fatigue and stiffness should also be considered.

The filler contents (material < 0.075 mm) of LAMBS must be controlled. Filler/binder ratios of LAMBS should not exceed 1.5 (by mass). The film thickness of LAMBS should be between 6 and 8 microns.

Guidelines for Component Selection:

Aggregate
The aggregates used in LAMBS should be durable and have high crushing strengths. The flakiness of the large aggregate fraction should be minimised (limited to 30 per cent) in order to improve mix stability and prevent segregation. Crushed aggregates are preferred as these promote aggregate interlock. It is recommended that at least 90 per cent (by mass) of the crushed aggregate should have two or more fractured faces. The angularity of the fine aggregate in a LAMBS mix should be given careful consideration, so that the stability of the mix can be optimised, on the one hand, and that its workability can be ensured, on the other. A minimum fine aggregate angularity of 45 per cent is recommended (Method A of AASHTO TP3320 or ASTM C1252-93). The use of rounded natural sand in LAMBS should be minimised or, if possible, avoided entirely. A minimum sand equivalent value of 45 is recommended, i.e. the ratio of the sand to clay height in the sand equivalent test, expressed as a percentage (AASHTO T176).

Binder
If required, the use of high viscosity binders (40/50 pen) may improve the stiffness and, hence, the load-spreading ability of LAMBS. Modified binders have been used to provide added stiffness. Adhesion of the binder to the aggregate is essential to prevent stripping.

Filler
As mentioned previously, the filler content (material < 0.075 mm) of LAMBS must be controlled. The use of fillers with a higher percentage of very fine particles (material < 10 microns) should be avoided. An active filler (hydrated lime) content of 1 per cent may enhance the adhesion properties of the binder and is strongly recommended for LAMBS.

Selection of a Design Gradation
By definition, LAMBS do not presuppose a specific gradation. For continuously graded mixes, however, the Fuller equation below is often used to design mixes for maximum density:

\[ P = \left( \frac{d}{D} \right)^n \]

where

- \( P \) = percentage passing sieve size \( d \) (mm),
- \( D \) = maximum stone size (mm),
- \( n \) = a parameter to determine the shape of the grading curve.

The maximum aggregate sizes used for LAMBS may be 37.5 mm or 53 mm. An \( n \)-power of 0.45 results in mixtures with minimum VMA. It should be noted that gradations that have
inadequate VMA often do not allow a sufficient amount of binder for durability and fatigue resistance. The use of the Fuller equation may result in LAMBS with excessive filler contents and for this reason, the gradations of continuously graded LAMBS often follow those for Dense Bitumen Macadams defined by the following gradation equation:

\[ P = \frac{(100 - F)(d^n - 0.075^n)}{(D^n - 0.075^n)} + F \]

where

- \( P \) = percentage passing sieve size \( d \) (mm),
- \( D \) = maximum stone size (mm),
- \( F \) = filler content (% by mass of aggregate),
- \( n \) = a parameter to determine the shape of the gradation curve

This equation is preferred, as the filler content of the mix may be set at a predetermined level. Gradation curves with \( n \)-values higher than 0.7 tend to segregate and should only be used in special circumstances. An \( n \)-value of between 0.4 and 0.7 is generally used.

There is limited information available on the gradation design of LAMBS which promote stone skeleton type mixes. The design principles of SMA type mixes should be applied.

**Design process**

The design process entails the following steps:

1. Determination of which grading/filler combinations are achievable with the material available from the quarry;
2. Preparation of laboratory samples using three achievable candidate gradings compacted at the estimated optimum binder contents of these gradings and (optionally) at two other binder contents;
3. Determination of density and calculation of the voids content, percentage voids in mineral aggregate, voids filled with bitumen and film thickness;
4. Rejection of any grading/filler/binder combinations which do not comply with the volumetric criteria and choosing an optimum aggregate blend;
5. Compaction of the optimum aggregate blend at four different binder contents and selection of a design optimum binder content in terms of the resulting volumetric properties;
6. Determination of the indirect tensile stiffness, indirect tensile strength and strain at maximum stress of the optimum blend at the various binder contents;
7. Rejection of those binder contents which result in mixtures which do not comply with the criteria for stiffness, indirect tensile strength and strain at maximum stress;
8. Dynamic creep testing of the remaining binder content combinations;
9. Rejection of those binder contents which result in mixtures which do not comply with the dynamic creep modulus criteria;
10. Selection of the optimum grading/filler/binder mixture, based on performance and behaviour, required constructibility aspects and cost considerations, and
11. Optionally, checking of the fatigue and durability aspects of the design mix.
Note that if the asphalt supplier proposes a suitable mix, steps 1 to 4 may be disregarded. The design process may be summarised as shown in Figure 4.9.

**Laboratory Compaction**

Because of its availability, the Hugo method is recommended for the laboratory compaction of 150 mm diameter LAMBS samples. The automated Hugo hammer is preferred. However, if manual compaction is used, steps should be taken to maintain consistency in compaction. As LAMBS are generally used for high traffic volumes and axle loadings, it is important that the mix design be done with the best available procedures. Provision should therefore be made for the gyratory compactor, where it is available. It is recommended that the final mix design, when designed using the Hugo method, be correlated with the gyratory compactor.

The 10,438 kg Hugo hammer is used to apply 110 blows to one side of the sample. The compaction energy applied is related to the volume of material required to achieve a compacted design height of 96 mm (a guideline at which the operator must aim) and deviation from this height requires the number of blows to be adapted to ensure that the same unit compaction energy per volume of material is applied. It should be noted that the applied energy is related to the volume of material when the compacted height is 96 mm. The unit energy compaction should be maintained if another design height is used. After each blow the hammer is rotated by one segment. The mould is then inverted and the process repeated.

**Design Criteria**

Design criteria for LAMBS as outlined in Sabita manual 13\textsuperscript{19} are shown in the following table.

**Table 4.5: Design criteria for LAMBS\textsuperscript{19}**

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>SPECIFIED LIMITS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Volumetric properties</strong></td>
<td></td>
</tr>
<tr>
<td>Percentage voids (Hugo)</td>
<td>4% min, 6% max</td>
</tr>
<tr>
<td>Density</td>
<td>Aim for maximum density</td>
</tr>
<tr>
<td>VMA (binder content)</td>
<td>Aim for dry side of minimum VMA vs binder curve</td>
</tr>
<tr>
<td>VMA (percentage)</td>
<td>12% (min) for 25 mm Nominal Maximum Size, 11% (min) for 37.5 mm Nominal Maximum Size</td>
</tr>
<tr>
<td>Vbe (percentage)</td>
<td>Aim for 65% (min), 75% (max)</td>
</tr>
<tr>
<td>Film thickness</td>
<td>Aim for 8 microns (min) (depends on aggregate type)</td>
</tr>
<tr>
<td><strong>Mechanical properties</strong></td>
<td>For stiff layer: 2 000 Mpa (min)</td>
</tr>
<tr>
<td>Stiffness @ 25° C/10 Hz</td>
<td>For flexible layer: 1000 MPa (min)</td>
</tr>
<tr>
<td>Stiffness @ 25° C/10 Hz</td>
<td>2500 Mpa (max)</td>
</tr>
<tr>
<td>ITS @ 25° C, 50 mm/min</td>
<td>800 kPa (min)</td>
</tr>
<tr>
<td>Dynamic creep modulus on laboratory briquettes 40° C, 0.5 Hz, 2 hours testing</td>
<td>15 Mpa (min)</td>
</tr>
</tbody>
</table>
Select three candidate gradings

Measure volumetric proportions and check against criteria and choose design aggregate structure

Example:
- Pb = 3.5%
- Pb = 4.0%
- Pb = 4.5%

Manufacture specimens using Hugo hammer or gyratory compaction at three binder contents

Mix as proposed by asphalt supplier

Manufacture specimens with design aggregate structure using Hugo hammer or gyratory compaction at four binder contents

Pb = Popt – 0.3%
Pb = Popt
Pb = Popt + 0.3%
Pb = Popt + 0.6%

Popt = Optimum binder content

Measure volumetric proportions and check against criteria

Measure mechanical properties and check against criteria

Pass
Spec
Fail
Reject

Provisional mix

Optionally

Check fatigue life
Check durability
Design mix

Figure 4.10 Design process
Performance testing

Indirect tensile testing is done to determine the indirect tensile strength (ITS), the strain at maximum stress and the resilient modulus (stiffness) of LAMBS. The dynamic creep test is used to assess the deformation characteristics of LAMBS. Sabita manual 13\textsuperscript{19} details the specifications for these tests. The design criteria are outlined in Table 4.5. Optionally, the fatigue characteristics and the moisture susceptibility of LAMBS may be determined. Four point bending or semi-circular bending (SCB) are recommended for fatigue testing. To assess the susceptibility of the mix to moisture damage, it is recommended that the procedure as outlined in ASTM test method D4867\textsuperscript{6} be followed. The tensile strength ratio (TSR) of the indirect tensile strength of conditioned samples to that of normal samples should be greater than 0.8 for the mix to be resistant to water damage under normal loading and environmental conditions. Procedures for the conditioning of LAMBS specimens for moisture susceptibility tests are outlined in Sabita manual 13\textsuperscript{19}. 

5. MEASUREMENT OF VOLUMETRIC PROPERTIES

5.1 Measurements needed to Determine Volumetric Properties

The volumetric mix design procedure described in Chapter 4 is based primarily on determination of density, voids in the mineral aggregate (VMA), voids in the total mix (VTM) and voids filled with binder (VFB). To measure and calculate these quantities, standard procedures have been established and are described in Standard Methods of Testing Road Construction Materials (TMH1: 1985). Table 5.1 shows a typical worksheet which suggests the order of measurement and calculation. This worksheet can easily be programmed into a spreadsheet application. The measured quantities needed to facilitate volumetric design are:

Component Properties (Steps 1 to 6 in Table 5.1)
- Bulk density, apparent density and absorption of coarse aggregate (TMH1:1985, Test Method B14)
- Bulk density, apparent density and absorption of fine aggregate (TMH1:1985, Test Method B14), and
- Relative density of bituminous binders (TMH1:1985, Test Method E2).

Density Properties of the Mix (Steps 7 to 10 in Table 5.1)
- Bulk specific gravity of compacted mix (TMH1:1985, Test Method C3);
- Maximum theoretical specific gravity of mix (TMH1:1985, Test Method C4(a)), and
- Percentage binder absorption (TMH1:1985, Test Method C4(b)).

In addition to the quantities described above, measurement of volumetric quantities also requires the percentage binder content to be known. This quantity is normally determined during the mix design process and is controlled by proportioning of mix components. TMH1:1985 also specifies methods for the determination of the binder contents of compacted mixes. Three methods are described:

i) Method C7(a): Direct method;
ii) Method C7(b): Indirect method, and
iii) Method C7(c): Reflux method.
Table 5.1: Procedure for Determination of Component Properties and Volumetric Quantities of Mix

<table>
<thead>
<tr>
<th>Step</th>
<th>Measured/Calculated Quantities</th>
<th>Symbol</th>
<th>Method/Formula †</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bulk relative density of coarse aggregate</td>
<td>Gsb1</td>
<td>B14 (TMH1: 19855)</td>
</tr>
<tr>
<td>2</td>
<td>Apparent relative density of coarse aggregate</td>
<td>Gsa1</td>
<td>B14 (TMH1: 19855)</td>
</tr>
<tr>
<td>4</td>
<td>Bulk relative density of fine aggregate</td>
<td>Gsa2</td>
<td>B15 (TMH1: 19855)</td>
</tr>
<tr>
<td>5</td>
<td>Apparent relative density of fine aggregate</td>
<td>Gsb2</td>
<td>B15 (TMH1: 19855)</td>
</tr>
<tr>
<td>6</td>
<td>Absorption of coarse and fine aggregate</td>
<td>Abs</td>
<td>B14 (TMH1: 19855) &amp; B15 (TMH1: 19855) **</td>
</tr>
<tr>
<td>7</td>
<td>Relative density of binder</td>
<td>Gb</td>
<td>E2 (TMH1: 19855)</td>
</tr>
<tr>
<td>8</td>
<td>Bulk relative density of total aggregate</td>
<td>Gsb</td>
<td>[ G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{P_1 G_{sb1} + P_2 G_{sb2} + \ldots + P_n G_{sbn}} ]</td>
</tr>
<tr>
<td>9</td>
<td>Maximum theoretical relative density (MTRD) of mix (Rice’s method) **</td>
<td>Gmn</td>
<td>C4(a) (TMH1: 19855) **</td>
</tr>
<tr>
<td>10</td>
<td>Bulk relative density of compacted mix</td>
<td>Gmb</td>
<td>C3 (TMH1: 19855) ††</td>
</tr>
<tr>
<td>11</td>
<td>Effective relative density of total aggregate</td>
<td>Gse</td>
<td>[ G_{se} = \frac{100 - P_b}{100 - \frac{P_b}{G_{mn}}} ]</td>
</tr>
<tr>
<td>12</td>
<td>Percentage absorbed binder</td>
<td>Pb</td>
<td>[ P_{ba} = 100 \times \frac{G_{se} - G_{sb}}{G_{sb} \cdot G_{se} \cdot G_b} ]</td>
</tr>
<tr>
<td>13</td>
<td>Effective binder content</td>
<td>Pbe</td>
<td>[ P_{be} = P_b - \frac{P_{ba}}{100} \cdot P_s ]</td>
</tr>
<tr>
<td>14</td>
<td>Voids in the Mineral Aggregate (VMA)</td>
<td>VMA*</td>
<td>[ VMA = 100 - \frac{P_s}{G_{mb}} \cdot \frac{G_{sb}}{G_{mn}} ]</td>
</tr>
<tr>
<td>15</td>
<td>Air void content in compacted mix</td>
<td>Va</td>
<td>[ V_a = 100 \cdot \frac{G_{mn} - G_{mb}}{G_{mn}} ]</td>
</tr>
<tr>
<td>16</td>
<td>Voids filled with binder</td>
<td>VFB</td>
<td>[ VFB = 100 \cdot \frac{VMA - V_a}{VMA} ]</td>
</tr>
</tbody>
</table>

* The equation for VMA assumes mix composition is determined as percent by weight of total mix;
† Symbols needed for calculations, not defined in Table 5.1:
\[ P_1, P_2, P_n = \text{percentages by mass of individual aggregate fractions}; \]
\[ G_{sb1}, G_{sb2}, G_{sbn} = \text{bulk relative densities of individual aggregate fractions}; \]
\[ P_b = \text{percentage binder content at which MTRD was determined}; \]
\[ P_s = \text{percentage aggregate content by mass of total mix}; \]
** Aggregates with absorption values above 1.5 per cent require special considerations for determination of MTRD.
†† An alternative method for measurement of the mix BRD has been developed and is described in Appendix A.
6. EVALUATION OF PERMANENT DEFORMATION \(^{22,23}\)

6.1 General

Permanent deformation, or rutting, is a complex phenomenon which poses significant challenges as far as performance evaluation is concerned. It is one of the most frequently observed and serious type of distress on hot-mix asphalt layers. Permanent deformation can lead to ponding of water in wheel tracks and can therefore be regarded as a serious road hazard in wet weather. Rutting can also lead to poor riding quality, which may result in increased vehicle operating costs. In South Africa, the frequency of rutting on flexible pavement structures has increased markedly over the past few years. Although this observation may partly be attributable to the high rainfall observed in some years, researchers and practitioners believe that an increase in traffic volumes, coupled with increasing axle loads and tyre pressures may also be contributory factors. Since increases in traffic volumes, vehicle loading and tyre pressures may be expected to continue, there is a definite need to adapt hot-mix design methods to ensure that asphalt mixes are sufficiently stable to accommodate these increases.

6.2 The Mechanism of Rutting

It is widely acknowledged that rutting is a two-phase process consisting of (i) densification accompanied by a decrease in volume and (ii) shear deformation at constant volume. Although these mechanisms can play a role at the same time, initial consolidation normally precedes shear deformation. For well-compacted asphalt mixes, shear deformation is believed to be the main contributor to permanent deformation. The characteristics of these two phases are as follows:

**Densification and Volume Decrease**

During the initial densification phase, the mix undergoes further compaction owing to the action of traffic. In this phase, aggregates are pushed into their preferred orientation positions and a decrease in air void content results from this re-orientation. As densification increases, the mix stability normally improves, resulting in the typical decrease in rate of deformation seen in the curves of repetitive loading versus permanent deformation.

During densification, the air void content of the mix may decrease from an initial value of 7 or 8 per cent and – in a well designed mix – will reach an equilibrium at approximately 4 per cent. The mix is typically designed to operate at this void content, at which shear resistance is expected to be optimal or satisfactory for the demands of traffic.

It should be noted that a continued application of traffic or compaction energy may tend to reduce the void content to a refusal density limit that may be as low as 1 per cent. However, for well designed mixes, this normally does not occur in the field, since the increased stability afforded by aggregate re-orientation and ageing of the binder largely prevents further densification.

Mixes for which the air void content decrease to below 3 per cent during the densification period are more prone to rutting than those mixes which stabilize at air void levels of approximately 4 per cent. The air void content attained after primary densification by traffic is an important design parameter, since it is a key determinant of the resistance to shear deformation.
Shear Deformation
Shear takes place when the combined resistance to deformation afforded by friction and cohesion is overcome by the imposed stress state. In the case of asphalt pavements, shear deformation consists of small flow movements associated with repetitive traffic loads. Aggregates are gradually pushed downward and/or sideways in small increments until a depression or rut is formed.

It should be clear that the movement associated with a single load application will be small and will be associated with movement or breakdown at particle-to-particle interfaces. The mechanism is thus one of attrition and slow movement rather than of sudden or large-scale movement of particles over one another. These movements are resisted by (i) the cohesion afforded by the mastic (binder and filler combination), (ii) the macro-interlock attained by the aggregate skeleton and (iii) the durability and frictional aspects of the aggregate skeleton, as determined by aggregate hardness, angularity and durability.

6.3 Environmental Aspects Related to Rutting Resistance

It should be clear from the above description of the mechanism of permanent deformation that rutting is primarily resisted by the cohesive and frictional elements of the mix. Although these two elements are often characterized respectively by the binder and aggregate components, the interaction that takes place between these two elements is in fact highly complex and non-linear. The relative degree of shear resistance afforded by the cohesive and frictional elements depend on a number of factors, the most important of which are discussed below.

Temperature
Creep tests on asphalt mixes suggest that temperature is the most influential variable affecting rutting behaviour. Temperature primarily affects the viscosity of the binder. At temperatures of approximately 45°C and higher, the binder softens considerably and shear resistance becomes highly dependent on the frictional resistance offered by the aggregate. The amount of cohesion that is still offered by the binder at these high temperatures is a function of the binder type. Some binders (notably polymer modified binders (PMB’s)) may still contribute considerably to the shear resistance at high temperatures.

At temperatures of approximately 30°C and below, the binder stiffens considerably so that most of the resistance to permanent deformation is derived from the cohesive component.

Loading Rate or Vehicle Speed
The rate of loading also influences cohesive resistance. Bituminous binders are viscoelastic in nature and therefore soften at high temperatures or at low rates of loading. Thus, at high vehicle speeds, the cohesive resistance increases, the converse being true at low vehicle speeds.

Stress State
The stress state associated with a particular loading and pavement situation can be characterized by two stress components: (i) the shear stress, which tends to distort the material and which leads to shear deformation and (ii) the bulk stress, which provides an indication of the degree of confinement afforded by the stress state. The bulk stress therefore adds to the rut resistance, while the shear stress is the primary cause of deformation.
At higher temperatures, the amount of permanent deformation resulting from a single load application is directly related to the magnitude of the imposed shear stress, and indirectly related to the degree of confinement afforded by the bulk stress. At lower temperatures, the influence of the stress magnitude becomes less prevalent. Thus, at high temperatures, a relatively small change in applied shear stress may cause a large increase in permanent deformation. At low temperatures the same change in stress state will have a much reduced effect on permanent deformation. This is because of the overriding effect of cohesion at low temperatures.

6.4 Mix Aspects Related to Rutting Resistance

**Viscosity of the Mastic**
The combination of binder and filler that makes up the mastic of an asphalt mix is a key determinant of rutting resistance. Two characteristics of the mastic play a role in determining rut resistance: (i) the viscosity of the mastic and (ii) the temperature sensitivity of the mastic. Binder-filler combinations with a high viscosity will increase the rut resistance of the mix. Binders with lower softening points will tend to lose more cohesive strength at high temperatures than those with higher softening points, thereby reducing the rut resistance at high temperatures.

It should be noted that the relative importance of the binder contribution to rut resistance is highly dependent on the skeleton type and on the effectiveness of frictional resistance, as determined by the gradation and aggregate characteristics.

**Packing Characteristics of the Mix**
The manner in which the aggregates are packed is one of the factors determining frictional shear resistance (the others being bulk stress and aggregate characteristics). A mix in which stone-to-stone contact is attained without overfilling of the voids (i.e. a stone skeleton mix) will generally have a greater frictional resistance than a mix in which the voids between the larger aggregates are filled with finer fractions (i.e. a sand skeleton mix).

**Volumetric Aspects**
Voids-filled-with-binder (VFB) has been shown to be one of the volumetric parameters with the strongest relationship to rutting performance. The extent to which the voids between large aggregates are filled with binder plays an important role in determining rut resistance. Although the binder contributes to the cohesive strength of the mix, the cohesive capability of many binders is greatly reduced at high temperatures. Thus, at high temperatures, aggregate interlock becomes increasingly important in determining rut resistance. A void structure that is overfilled with binder will tend to lubricate aggregates (or even force them apart), thereby reducing frictional resistance, with a resulting increase in rutting potential. There is some evidence to indicate that aggregate structures tend to become overfilled with binder as the void contents approach 2 per cent.

**Aggregate Characteristics**
Aggregate angularity, hardness and durability play an important role in determining frictional shear resistance. Even if a dense packing with stone-to-stone contact is achieved, the frictional resistance will be poor if the aggregates are rounded or tend to become rounded or fractured after many load applications.
6.5 **Evaluation of Resistance to Permanent Deformation**

To date, no single simplified test has been developed which was proven to provide a consistent evaluation of rutting performance for all mix types. Of the available tests, wheel tracking tests appear to have the strongest correlation with rutting in the field. However, wheel tracking tests are relatively expensive to perform and require large slabs of material to be compacted before testing.

To overcome this problem, three alternative methods for rutting evaluation were developed:

i) Expert system approach for densely graded sand-skeleton mixes;

ii) Axial loading slab test;

iii) Wheel tracking test;

An overview of the different approaches is given in the following sections.

6.6 **Expert systems Approach to Rutting Evaluation (Sand-Skeleton Mixes)**

This approach is recommended for the evaluation of the rutting potential for low volume roads or applications with a low design objective rating for rutting (e.g. applications in colder regions). This approach can also be implemented for basic quality control to evaluate rutting potential during construction. Although the expert system approach is highly simplified, it is believed to be an improvement on the Marshall stability and flow test, which was often in the past used as the only test to assess mix stability for low to medium level traffic applications.

The purpose of the expert system is not to predict rutting in absolute terms, but merely to alert designers to situations in which rutting potential can be high. The approach relies primarily on simple tests, including the evaluation of simple component and volumetric test indicators to provide an indication of the cohesive and frictional strength components as well as of the overall rutting potential for a given traffic and climatic situation.

The system consist of two tables, together with a simple weighting and aggregation system. The aggregation process results in a parameter which allows a designer to evaluate the rutting potential of a mix based on several inexpensive tests.

In this system the test information is evaluated against the design situation to provide an overall indication of rut potential. The evaluation of relative cohesive and frictional strength is based the observed distribution of values from test parameters on a range of materials. This evaluation is coupled with published empirical knowledge of threshold values for the different test parameters beyond which rut potential can be expected to increase significantly.

Appendix D contains the relevant tables and some explanation on how the values contained in the tables were derived. Suggestions for implementation are also offered.
6.7 Axial Loading Slab Test

The axial loading slab (ALS) test with the associated probabilistic analysis is recommended for high volume roads or applications where a level 3 rating is obtained for the rutting as a design objective.

The test involves a small slab of asphalt compacted to the specified field density by means of a slab compactor or extracted from the field. The slab is placed in a steel mould, and on a synthetic support with a known stiffness. Three types of synthetic materials (representing low, medium and high stiffness supports) can be selected to approximate the support conditions for the planned asphalt layer. The asphalt slab is repetitively loaded in the axial direction using a loading platen with a 100 mm diameter. The test is performed at three temperature levels.

The ALS test data can be used to develop a regression equation which estimates permanent deformation as a function of number of load applications, temperature and stress level. This equation can be used to evaluate the mix rut potential at different loads and temperatures. Such an analysis can be performed using a computer spreadsheet. Ideally, the ALS data and regression equation should be used in a Monte Carlo simulation to estimate the development of permanent deformation under cumulative traffic. Computer programs such as the PRORAS (Probabilistic Rut Analysis System) computer program can be used to perform the Monte Carlo analysis.

The ALS test and associated PRORAS analysis system were developed to simulate field conditions as closely as possible as far as rate of loading, stress state, temperature and traffic load distribution is concerned. The probabilistic analysis takes into account the variation in loading due to random variations in temperature and load as well as traffic wander. In addition, both the daily and monthly temperature variation is taken into account in the analysis. The PRORAS system therefore provides a quantified estimate of actual rutting performance. However, because the laboratory test is accelerated and scaled down, the results still need to be interpreted in a somewhat relative manner, and comparison with similar mixes tested in the past is recommended.

6.8 Wheel Tracking Tests

Although wheel tracking tests appear to be well correlated with rutting in the field, there are at present no quantified relationships to link wheel tracking test results to rutting in the field under variable traffic loading and environmental conditions. For this reason, wheel tracking tests cannot be used to provide a quantitative estimate of rutting in the field. The test does, however, provide a reliable estimate of the rutting potential of a mix relative to similar mixes that have been tested in the past. Wheel tracking tests are particularly recommended for the evaluation of rutting performance on stone skeleton mixes, or mixes which involve modified binders, as experience has shown that these mix types cannot be properly evaluated by means of conventional test methods such as the dynamic creep and ITS. Currently, two types of wheel tracking devices are used for rutting evaluation in South Africa:

Model Mobile Load Simulator (Mk.3 MMLS)

The MMLS is operated and distributed by the University of Stellenbosch. The MMLS differs from most wheel tracking devices in that trafficking is achieved with 4 bogies instead of a single wheel
(see Figure 6.1). Each bogie consists of a single 300 mm diameter wheel, with a maximum inflation pressure of 800 kPa and a maximum load of 2,7 kN.

A major advantage of the MMLS, as compared to other wheel tracking devices, is its high rate of trafficking: more than 10,000 simulated axle loads per hour can be applied. Another significant advantage of the MMLS is that it can be transported to field sites for the testing of full scale asphalt pavements. The weight of the Mk.3 MMLS is approximately 800 kg (including dead weight needed for 2,4 kN load per wheel).

Transportek Wheel Tracking Device
The Transportek wheel tracking device was developed to assess the rutting susceptibility of asphalt mixes and also to enable the measurement of strains under moving wheel loads. The device is used both to compact and test asphalt slabs. A segment of a steel wheel roller is used for compaction, while a solid rubber wheel (400 mm diameter, 100 mm wide) is used for rutting evaluation. Slab dimensions can be 280 by 320 mm or 350 by 660 mm, with the latter slab size being used most often for rutting evaluation. Figure 6.2 shows the compaction of a slab before testing. Figure 6.3 shows the wheel tracking device in operation, and Figure 6.4 shows a transverse section of a slab after wheel track testing.
Figure 6.2 Compaction of Slab for Wheel Tracking Test

Figure 6.3 Wheel Track Testing
Typically, the level of compaction aimed for in the wheel tracking test will be the same as that which is specified for field compaction. The level of compaction achieved during the compaction process can be accurately controlled with the Transportek wheel tracking device. However, the actual density achieved is dependent on the accuracy and applicability of the MTRD and mass calculations used to determine the amount of material that should be used to compact a slab. It is therefore recommended that more than one slab be compacted for the monitoring of the voids and density that is achieved after slab compaction. If the wheel tracking tests are carried out at the design stage, it is imperative that the MTRD of the design mix and that of the eventual plant mix be compared to ensure that the rut performance of the constructed mix is comparable to that obtained during wheel tracking tests. Ideally, the plant mix should also be tested in the wheel tracking device to ensure that the properties of the design mix are the same as those of the plant mix.

Because wheel tracking test results are evaluated in a relative manner, data post-processing is minimal and therefore the cost of testing is significantly lower than that of the axial loading slab test. The test is therefore recommended for applications where rutting has a medium to high importance as a design objective, but which do not justify the higher cost associated with a quantitative estimate of rutting performance.

The standard test protocol for the Transportek wheel tracking device is to perform the test at 60°C and at a load of 600 kg (which equates to a contact pressure of approximately 900 kPa). For this test protocol, the limits shown in Table 6.1 can be used as a tentative guideline to the evaluation of rutting performance.

<table>
<thead>
<tr>
<th>Repetitions to 10 mm Rut Depth</th>
<th>Mix Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 2500</td>
<td>Poor</td>
</tr>
<tr>
<td>2500 - 5000</td>
<td>Medium</td>
</tr>
<tr>
<td>&gt; 5000</td>
<td>Good</td>
</tr>
</tbody>
</table>
6.9  Recommended Test Procedure for Rutting Evaluation

The three approaches to rutting evaluation provide designers with a flexible and cost effective approach to rutting evaluation which can be adapted to suit most design situations. Designers should become familiar with the advantages and disadvantages of the different rut evaluation procedures so that the most cost effective and appropriate evaluation method can be selected for a particular design situation. Table 6.2 provides a recommended test selection matrix for different design situations.

Table 6.2 Recommended Rut Evaluation Tests

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Design Objective Rating for Rutting</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td><strong>Sand Skeleton Mixes</strong></td>
<td></td>
</tr>
<tr>
<td>Dense Graded Mixes</td>
<td>Expert System Evaluation</td>
</tr>
<tr>
<td>LAMBS</td>
<td></td>
</tr>
<tr>
<td><strong>Stone Skeleton Mixes</strong></td>
<td>Spatial composition</td>
</tr>
<tr>
<td>Stone Mastic Asphalt</td>
<td></td>
</tr>
<tr>
<td>Open Graded Mixes</td>
<td></td>
</tr>
</tbody>
</table>
7. EVALUATION OF FATIGUE PERFORMANCE

7.1 General

As fatigue cracking in asphalt mixes is a major form of distress, mixes should be evaluated during the mix design stage to assess their fatigue performance. The evaluation of fatigue performance is complicated by the interaction between the asphalt layer properties, pavement structure and the environment. The fatigue performance of an asphalt mix therefore cannot be evaluated in isolation, but has to be assessed in relation to the conditions of its immediate support. Chapter 2 contains guidelines on proper support conditions for different traffic scenarios.

In the past, attempts were made to quantify fatigue performance of asphalt mixes by using logarithmic curves relating strain level to number of load repetitions before cracking occurred. Strain is typically calculated by means of multilayer elastic models in which a fixed load and support condition is used to characterize the strain behaviour under traffic loading. A shift factor is then applied to convert the number of repetitions to failure obtained in the laboratory to that expected in the field.

Recent research suggests that this approach greatly oversimplifies the fatigue phenomenon and may lead to misleading interpretation of fatigue potential. Some reasons for this observation are:

Conventional multilayer elastic models significantly overestimate the strains which take place in asphalt layers under dynamic loading;

Unavoidable variations in support condition, traffic axle configurations and wander, traffic speeds, tyre pressures and axle loads can lead to a significant variation in actual tensile strains for a uniform section of road. It is therefore not appropriate to represent the strain condition in a road section by a single strain value. It is more appropriate to calculate a range of expected strains using a probabilistic response model, and then to characterize the working strain range of the pavement as low, medium or high, as explained in the following sections.

Temperature significantly affects the fatigue life of asphalt mixes. The fatigue life at 20°C can be as much as five times higher than that tested at 5°C. It is therefore not realistic to estimate the fatigue life of an in-service pavement - whose daily working temperatures can range from 15°C to 60°C - on the basis of a laboratory fatigue test performed at 5°C.

Rest periods between load applications can significantly influence the fatigue life of asphalt mixes. The shift factor, which is normally applied during mechanistic design should account for the influence of rest periods. However, research suggests that this influence is not constant, but depends on the working strain range. It is therefore not reasonable to assume that a constant shift factor can be applied to all design situations.

Fatigue cracks often initiate from the top of the asphalt layer; not from the bottom, as is assumed in conventional mechanistic analysis. This phenomenon has not been properly explained and suggests that the conventional approach to fatigue evaluation may be inappropriate for many design situations.
Because of the many deficiencies in the current method of fatigue life prediction, a shift in approach is advocated in the new HMA design method. In this approach, the emphasis is placed not so much on the prediction of fatigue life in absolute terms, but rather on proper evaluation of the relative fatigue performance of the design mix.

### 7.2 Performance Testing for Fatigue Evaluation of Asphalt Wearing Courses

Apart from the four point bending beam fatigue test, very few laboratory tests provide a consistent evaluation of the fatigue performance of wearing courses. As four point bending beam fatigue tests are relatively expensive to perform, they are therefore mostly used for applications on high trafficked roads. For the design of wearing courses on pavements with low traffic volumes, where expensive performance tests are not always warranted, the indirect tensile strength (ITS) test parameters, coupled with the evaluation of binder durability, can be used to determine whether there is a risk of premature fatigue failure and whether four point bending beam fatigue tests are warranted. Table 7.1 provides guidelines for the interpretation of ITS data (measured at 25°C) for the fatigue evaluation of asphalt wearing courses.

#### Table 7.1 Guidelines for the Interpretation of ITS results for fatigue performance evaluation

<table>
<thead>
<tr>
<th>Relative Fatigue Performance</th>
<th>ITS (kPa)</th>
<th>ITS strain at Maximum Stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good</td>
<td>&lt; 1000</td>
<td>&gt; 2.2</td>
</tr>
<tr>
<td>Medium</td>
<td>1000 to 1400</td>
<td>1.5 to 2.2</td>
</tr>
<tr>
<td>Poor</td>
<td>&gt;1400</td>
<td>&lt; 1.5</td>
</tr>
</tbody>
</table>

Note: The above recommendation is only valid for relatively thin wearing courses. For asphalt base courses, the ranking would be the opposite of that shown above, where the designer should strive to achieve high ITS values.

Wearing course mixes for which Table 7.1 suggests a poor fatigue performance should be further evaluated by means of a detailed binder evaluation and an evaluation of the spatial composition. In such instances, designers may also consider performing bending beam fatigue tests to validate the indication given by the less exact ITS test.

### 7.3 Four Point Bending Beam Test

Figure 7.1 shows a schematic representation of the four point bending beam fatigue test. During the test, rectangular beam specimens are subjected to a repeated load. Tests are normally performed at 5°C, using a sinusoidal load with a frequency of 10 Hz. Two modes of loading can be applied. In the constant strain mode of loading, the strain at the bottom of the beam is preselected and kept constant for the duration of the test. Failure is defined as the point at which the stiffness of the beam is reduced to 50 per cent of its initial stiffness. In the constant stress mode of loading, the load is kept constant and the strain is allowed to vary. The test is terminated when the beam fails (i.e. cracks). The constant strain test is normally used for thin (< 70 mm) surfacing applications, the constant stress mode being used for the evaluation of thick (>70 mm) asphalt bases.
DISPLACEMENT MEASUREMENT USING LVDT

LOAD FRAME

TEST SPECIMEN

SERVO-HYDRAULIC ACTUATOR

LOAD CELL

ROTATIONAL BEARING

TRANSLATIONAL BEARINGS

Figure 7.1 Schematic Illustration of the Four Point Bending Beam Test

Figure 7.2 Compacted Sample using Rolling Wheel Compaction
Owing to the small strains induced during testing, as well as the dynamic nature of the loading, bending beam tests require sophisticated loading and data acquisition equipment. For this reason, bending beam fatigue tests are normally performed in specialist laboratories.

Bending beam samples are cut from a larger slab of material which is normally prepared using a rolling wheel compactor (see Figure 7.2) with the compaction effort and quantity of material being controlled so as to allow the design voids content to be attained. Approximate dimensions of beam samples are: 400 mm long x 60 mm wide x 50 mm high. Samples are preconditioned at the testing temperature by placing them in an oven set to the test temperature, several hours before they are tested.

7.4 Interpretation of Results

Table 7.2 provides guidelines for the interpretation of four point bending beam fatigue results. It should be noted that the limits shown in Table 7.2 pertain to constant strain tests performed at 5°C and at a loading rate of 10 Hz. In these tests, the point of failure was defined as the number of load repetitions at which the original beam stiffness had decreased by 50 per cent. The repetitions to failure referred to in Table 7.2 cannot therefore be interpreted as the number of axles before the onset of fatigue cracking.

<table>
<thead>
<tr>
<th>Relative Fatigue Performance</th>
<th>Number of Repetitions to Failure for Strain Regime (Millions) @ 5°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Strain (180 to 230)</td>
<td>Medium Strain (320 to 370)</td>
</tr>
<tr>
<td>Good</td>
<td>&gt; 2.4</td>
</tr>
<tr>
<td>Medium</td>
<td>1.0 to 2.4</td>
</tr>
<tr>
<td>Poor</td>
<td>&lt; 1.0</td>
</tr>
</tbody>
</table>

In an HMA design, Table 7.2 can be used as follows:

Step 1: The designer should calculate the expected strain level for the given design situation. Conventional multilayer elastic models can be used for this calculation. The calculated strain level is used to determine the approximate working strain regime for the design situation, as defined in Table 7.2.

Step 2: Perform four point bending beam tests at a strain level which falls inside the strain range shown in Table 7.2 for the appropriate strain regime. For example, for the low strain regime, tests can be performed at 220 microstrain. It is recommended that a minimum of 3 beams be tested at the appropriate strain regime.

Step 3: Use the bending beam test results together with the ranges shown in Table 7.2 to evaluate the relative fatigue performance of the design mix. Use Table 7.3 to estimate the design fatigue life of the design mix.
As can be seen from Table 7.3, the fatigue life increases dramatically if the working strain limit is below approximately 230 microstrain. For designs for high traffic volumes, it is therefore strongly advised that the recommended support conditions (see Chapter 2) be observed to ensure that the mix will operate in the low strain regime.

Mixes that operate in the low strain regime are likely to fail because of climatic influences (embrittlement due to oxidation etc.) rather than traffic loading. The fatigue life is thus likely to be coupled to time rather than to the number of load applications. Experience indicates that a typical effective fatigue life of thin asphalt mixes is between 8 and 12 years. Mixes with a good bending beam fatigue performance can be expected to last to the upper limit of this range, while those with a poor fatigue performance may last only to the lower limit.

Designers should be aware that mixes that operate in the medium to high strain regimes are very sensitive to variations in load and support conditions. It is recommended that mixes which are designed for these conditions, and which require a low risk of failure, be improved either by modification with bitumen rubber, or by increasing the binder content to ensure greater flexibility and durability.

Table 7.3 Guidelines for Fatigue Life Estimation (constant strain)

<table>
<thead>
<tr>
<th>Relative Fatigue Performance*</th>
<th>Approximate expected fatigue life before the onset of fatigue cracking</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low Strain (180 to 230)</td>
</tr>
<tr>
<td>Good</td>
<td>&gt; 15 MESA** or 12 years</td>
</tr>
<tr>
<td>Medium</td>
<td>8 to 15 MESA or 10 years</td>
</tr>
<tr>
<td>Poor</td>
<td>&lt; 8 MESA or &lt; 8 years</td>
</tr>
</tbody>
</table>

* As determined by bending beam fatigue tests and through Table 7.2.
** MESA = Millions of Equivalent Standard Axles.

7.5 Fatigue Evaluation of HMA Bases

Fatigue testing for HMA bases is normally performed in the constant stress mode of testing. To date, no constant stress tests using modern testing equipment have been performed in South Africa. Consequently guidelines for the interpretation of constant stress test results cannot be provided at this stage. Until more experience is gained in the interpretation of constant stress data, it is recommended that HMA base mixes be tested in the constant strain mode, at a strain level of 220 microstrain. The fatigue performance of the mix can then be evaluated in a relative manner using Table 7.2. Although this approach does not provide a reliable estimate of mix fatigue life, it may alert designers if the mix is likely to have unusually low fatigue properties.
8. OTHER DESIGN AND PERFORMANCE TESTS

8.1 Indirect Tensile Strength (ITS) Test

Test Description
The indirect tensile strength (ITS) test is commonly used to evaluate the cohesive strength of asphalt mixes. This property can be used to evaluate tensile strength (related to toughness and durability) and is also an important component of rutting resistance in the medium temperature range. The test does not require sophisticated testing equipment and can be performed on briquettes manufactured in the laboratory, as well as on cores obtained from the field.

In the ITS test the sample is loaded on its diametral axis, as illustrated in Figure 8.1. Figure 8.2 shows a sample positioned in the testing frame. The widths of the loading strips are prescribed by ASTM D4123 and are 13 mm for a 102 mm diameter specimen and 19 mm for a 152 mm diameter specimen.

During testing, the sample is loaded at a fixed rate of loading (a rate of 50 mm per minute is typically used) until a significant loss in applied load is noted. The peak load is used to calculate the indirect tensile strength. The formula for calculation of the ITS (in kPa) is as follows:

\[ \text{ITS} = \frac{2.0 \ P_{\text{ult}}}{\pi \ t \ D} \quad \text{Eq. 8.1} \]

Where:
- \( P_{\text{ult}} \) = Ultimate applied load, in kN;
- \( t \) = Thickness of the specimen, in mm, and
- \( D \) = Diameter of the specimen, in mm.

Figure 8.1 Schematic illustration of the Indirect Tensile Test
ASTM D4123\textsuperscript{6} prescribes sample dimensions and preparation for the indirect tensile strength and resilient modulus test. Specimens for the ITS test can be prepared using laboratory compaction techniques such as the Marshall or Gyratory compaction devices. Specimens can also be obtained from the field by means of coring. Specimens should have a thickness of at least 51 mm and a diameter of 102 mm or larger for aggregates of up to 25 mm maximum size. For aggregates with a maximum size of between 25 and 38 mm, the minimum thickness and diameter should be increased to 76 mm and 152 mm, respectively. Care should be taken to ensure that cores have smooth parallel surfaces.

**Interpretation of Results**

The indirect tensile strength of asphalt mixes provides an indication of the cohesive strength of asphalt mixes and is therefore strongly influenced by the characteristics of the binder. As an indicator of mix cohesion, ITS values provide an overall indication of mix stability in the low to mid-temperature range (10 to 40°C) and can be expected to be related to rutting resistance as well as durability and stripping potential.

The minimum value for ITS in South Africa is 800 kPa. However, designers should be aware that limited field studies have suggested that rutting potential tends to increase for ITS values below approximately 1000 kPa. At the same time, ITS values in excess of 1700 kPa may indicate a tendency to brittleness and low flexibility. An ideal range for ITS values would seem to be between 1100 and 1500 kPa. Table 8.1 shows some statistical parameters that are typically associated with ITS values. The comments in Table 8.1 refer to relatively thin asphalt wearing courses.
Table 8.1: Typical ITS Results*

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Typical ITS results (in kPa) at 25°C</th>
<th>Comments/Interpretation**</th>
</tr>
</thead>
<tbody>
<tr>
<td>95th percentile value</td>
<td>1650</td>
<td>Values above this may be indicative of brittleness or poor flexibility of wearing courses</td>
</tr>
<tr>
<td>75th Percentile Value</td>
<td>1200</td>
<td>Values close to this are indicative of good rutting performance</td>
</tr>
<tr>
<td>Average</td>
<td>1100</td>
<td>None</td>
</tr>
<tr>
<td>25th Percentile Value</td>
<td>900</td>
<td>Values below this may be indicative of poor rutting or stripping performance</td>
</tr>
</tbody>
</table>

* Based on 33 observations.
** SMA mixes and mixes manufactured with some polymer-modified or bitumen-rubber binders may have low ITS values and still exhibit good performance. The comments and interpretations may therefore not be applicable for such mixes.

8.2 Resilient Modulus (ASTM D4123)

Test Description
The indirect tensile test (ITT) is the most commonly used method for determining the resilient modulus of asphalt material. The test is relatively easy to perform and, most importantly, it can be performed on field cores. The sample dimensions and test configuration for the indirect tensile test is identical to that of the ITS test. However, the ITS test requires a loading frame which can apply a repeated dynamic load pulse of varying frequencies and durations. The equipment needed for the ITT test is therefore more sophisticated and expensive than that required for the ITS test.

The deformation is measured vertically (in the direction of loading) and horizontally by $v$ and $h$, respectively, as indicated in Figure 8.1. These deformations can be measured in two ways. ASTM 4123 prescribes a protocol for testing. According to this protocol, deformation is measured on the outside of the specimen. More recent developments include adaptations to measure deformation in the centre of the specimen by attaching a measurement device to the face of the specimen. Loading during resilient modulus testing is typically in the form of a haversine load pulse with a 0.1 second duration, followed by a 0.9 second rest period.

Figure 8.3 illustrates the stress state which is generated during indirect tensile testing. A relatively uniform tensile stress is generated in the horizontal direction along the central axis of loading. At the point where the loading strips make contact with the test specimen, this stress becomes compressive. The stress in the vertical direction is compressive all along the central loading axis but diminishes in magnitude towards the centre of the specimen.
The resilient modulus is determined as follows:

\[
Mr = \frac{P}{\delta_h + \frac{0.27 + \nu}{t}}
\]  
Eq. 8.2

where:

\[
\begin{align*}
Mr &= \text{Resilient modulus (Mpa)}; \\
P &= \text{Applied load, (N)}; \\
h &= \text{Horizontal deformation (mm)}; \\
t &= \text{Thickness of specimen (mm), and} \\
\nu &= \text{Poisson’s ratio.}
\end{align*}
\]

Poisson’s ratio can either be assumed (typical values are 0.25 at 5 °C and 0.40 at 40°C) or it can be calculated from the vertical and horizontal deformation. If the latter procedure is adopted, the following formula should be used to calculate Poisson’s ratio:

\[
\nu = 3.59\left(\frac{\delta_h}{\delta_v}\right) - 0.27
\]  
Eq. 8.3

Where \( \delta_v \) is the vertical deformation measured along the axis of loading.

Before the indirect tensile test can be performed, the applied load or stress level has to be determined. This is done by predetermining the Indirect Tensile Strength (ITS) of the mix (described in Section 8.1). The indirect tensile test for determination of Mr is then performed at a stress level of between 5 and 40 per cent of the ITS. Since loading during ITT testing is...
repetitive, higher stress levels should be avoided to prevent the specimen being damaged. The stress level should, however, also be sufficiently high to ensure that sufficient deformation is generated in the horizontal direction to allow accurate measurements to be made.

The resilient modulus is normally determined after 80 repetitive loading cycles have been applied. The load and deformations are then recorded for cycles 80 to 84 and Mr is determined for each of these cycles. The reported value for Mr is normally the average of these 5 cycles.

Sample preparation and dimensions are identical to that described for the indirect tensile strength test (Section 8.1)

8.3 Moisture Sensitivity (Modified Lottman Test)

Test Description
The Modified Lottman test for measurement of moisture sensitivity relies on indirect tensile strengths measurements taken before and after conditioning by freeze-thaw cycles. The test is performed according to the ASTM D4867 protocol (note: an alternative method is provided in AASHTO 283, but ASTM D4867 is preferred). In the test, six samples are compacted to within a void content range of 6 to 8 per cent (or to the field voids) and partially saturated with water (saturation limit of between 55 and 80 per cent). Three of the six samples are frozen for at least 15 hours and subsequently immersed for 24 hours in a hot bath set at 60°C (i.e. “conditioned” samples). All six samples are then brought to a constant temperature and their indirect tensile strengths determined (cf. Section 8.1). The ratio of the indirect tensile strengths of the conditioned and unconditioned samples is referred to as the tensile strength ratio (TSR).

Interpretation of Results
For routine mix design purposes, a minimum TSR of 0.7 is usually specified. For mixes in high rainfall areas and high traffic applications, a minimum TSR of 0.8 is recommended. Table 8.2 provides TSR criteria based on the permeability of the mix and the climate in which the mix will operate.

Table 8.2: TSR criteria based on mix permeability and climate

<table>
<thead>
<tr>
<th>Climate</th>
<th>Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
</tr>
<tr>
<td>Dry</td>
<td>0,60</td>
</tr>
<tr>
<td>Medium</td>
<td>0,65</td>
</tr>
<tr>
<td>Wet</td>
<td>0,70</td>
</tr>
</tbody>
</table>

8.4 Dynamic Creep Test for Evaluating Rutting Potential

Test Description
In the dynamic creep test, a cylindrical test specimen is subjected to repeated dynamic loads in the axial direction and the accumulated permanent deformation is monitored as a function of the number of load repetitions. In South Africa, a square wave load shape with a duration of 1 second...
and a rest period of 1 second is typically used. The applied load is typically 100 kPa and the test temperature is 40°C.

The test parameter used to evaluate dynamic creep results is the dynamic creep modulus, which is defined as:

\[
\text{Dynamic Creep Modulus} = \frac{\text{Applied Stress}}{\text{Permanent Strain}}
\]  

Eq. 8.4

where permanent strain is defined as the strain that accumulates between 30 and 3600 load applications.

The permanent strain that develops during the first 30 load applications is therefore subtracted from the total permanent deformation after 3600 cycles. This is to compensate for surface irregularities and to allow a ‘settling in’ period at the start of the test.

The dynamic creep test can be performed on compacted briquettes or field cores. Sample diameters are 100 or 150 mm, the sample heights for specimens prepared in the laboratory typically being between 60 and 100 mm.

Interpretation of Results

In recent years, research work has raised some doubts concerning the ability of the dynamic creep test to properly and consistently evaluate the rutting potential of different mix types. One of the main criticisms of the test concerns the absence of a confining pressure as well as the apparent insensitivity of the test results to low void contents. The test is generally regarded as being inappropriate for evaluating mixes that rely on stone-to-stone contact to develop rutting resistance.

For these reasons, the use of the dynamic creep modulus as an acceptance criterion is not recommended for mixes other than densely graded sand-skeleton mixes manufactured with unmodified binders. Mixes designed for situations which require superior rutting resistance should rather be evaluated using a wheel-tracking test. However, the dynamic creep test can be used in conjunction with volumetric test data and other performance tests to serve as a general check on the overall rutting potential of a mix (cf. Appendix D). Unusually low dynamic creep modulus values should alert designers to potential rutting problems and may prompt a re-evaluation of the volumetric design or may highlight the need for wheel tracking tests.

Table 8.3 contains some typical values for the dynamic creep modulus of different mixes. Designers should note that dynamic creep test results are often extremely sensitive to small variations in measured strains and applied loads. This applies specifically to mixes with high dynamic creep values, for which the overall permanent deformation is small enough to be affected by the precision of the loading and measurement devices.

Care should therefore be taken not to over-interpret small differences in dynamic creep moduli, specifically for high values (i.e. values approximately above 30 MPa). At least three replicates should be evaluated to determine whether the repeatability is acceptable. As a general rule it is recommended that the coefficient of variation should not exceed 20 per cent for samples prepared in the laboratory as part of the mix design process.
8-7

Table 8.3: Typical Values for Dynamic Creep Modulus*

<table>
<thead>
<tr>
<th>Expected Rutting Resistance</th>
<th>Dynamic Creep Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>Medium to Low</td>
<td>10 to 15</td>
</tr>
<tr>
<td>Medium to High</td>
<td>15 to 30</td>
</tr>
<tr>
<td>High</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>

*Stone-skeleton mixes and mixes manufactured with some polymer-modified or bitumen-rubber binders may have low dynamic creep values and still exhibit good performance. This test may therefore not be applicable for such mixes.

8.5 Cantabro Abrasion Test

Test Description

Basic Procedure
The Cantabro test is used to evaluate the abrasion resistance of open-graded mixes. The test is performed using the equipment for the Los Angeles abrasion test (ASTM C131-81®), but without any steel balls in the drum. A briquette is compacted using 50 blows with a standard Marshall hammer on each side. The mass of the specimen is determined to the nearest 0.1 gram, and is recorded as \( P_1 \). The test specimen is then placed in the Los Angeles drum at an operating temperature of 25°C. The drum is allowed to rotate for 300 revolutions at a speed of 30 to 33 rpm, after which the specimen is removed and the mass is again determined to the nearest 0.1 gram (\( P_2 \)). The percentage abrasion loss (\( P \)) is calculated as:

\[
P = 100 \times \frac{P_1 - P_2}{P_1} \quad \text{Eq. 8.5}
\]

Ageing of Specimens

For open-graded mixes, the Cantabro test procedure is performed both before and after simulated ageing. The procedure described above applies to unaged specimens. For tests on aged specimens, the following conditioning process is used:

Five briquettes are prepared at the optimum binder content as outlined above. The specimens are placed in a forced draft oven at 60°C for 48 hours. After this time, the temperature is raised to 107°C for a further 120 hours. After this oven ageing period, the specimens are removed from the oven and placed in a temperature cabinet at 25°C for four hours, after which testing is performed as described above.

Interpretation of Results

Unaged Specimens

The recommended maximum allowable abrasion loss for freshly compacted specimens is 20 per cent for any one of the five specimens. In some European countries a maximum allowable value of 25 per cent is specified.
**Aged Specimens**

The average abrasion loss for the five specimens should not exceed 30 per cent and no individual result should exceed 50 per cent.

The abrasion resistance of open-graded mixes tends to improve as the binder content increases, and is also related to the rheological properties of the binder. Open-graded mixes prepared with polymer-modified binders generally have better abrasion resistance than those prepared with conventional binders. Mixes prepared with bitumen rubber generally have the best abrasion resistance.

### 8.6 Schellenberg Drainage Test\(^{24}\)

**Test Description**

The Schellenberg drainage test is used to evaluate the binder run-off potential of open-graded and SMA mixes. The test procedure is simple and consists of placing 1000 to 1100 grams of uncompacted mix in an 800 ml glass receiver. The glass receiver is then placed in an oven set to the appropriate mixing temperature.

After a period of 1 hour ± 1 minute, the glass receiver is removed and emptied by turning it upside down. The glass receiver should not be shaken or vibrated. The material retained in the receiver is weighed and the percentage weight loss is determined.

**Interpretation of Results**

A weight loss of less than 0.2 per cent is considered good. A loss of between 0.2 and 0.3 per cent is acceptable and a weight loss of more than 0.3 per cent is considered poor and should prompt corrective action.

### 8.7 Axial Loading Slab (ALS) Test\(^{23}\)

The basic configuration of the ALS test is illustrated schematically in Figure 8.4. The test involves a relatively small slab of material, which can be extracted from the field or compacted using the TRL slab compactor. In the latter case, the compaction density should approximate the specified field density. The slab is then placed in a steel mould on a synthetic support of known stiffness. Three types of synthetic materials (representing low, medium and high stiffness supports) can be selected to approximate the support conditions for the planned asphalt layer.

The test protocol involves the use of three slabs, each of which is tested at a different temperature. Test temperatures are typically 60°C, 40°C and 30°C. At each temperature, the slab is subjected to 10 000 load repetitions at each of three different stress levels. The slab is loaded in the axial direction using a loading platen with a 100 mm diameter. Three stress levels are used (typically 1000 kPa, 700 kPa and 400 kPa), so that a total of 30 000 load repetitions are applied to the slab. The deformation is measured and recorded with increasing load repetitions.

The load is applied in a haversine waveshape. The load duration is determined on the basis of the expected heavy vehicle speed. For applications such as on climbing lanes and at intersections, where heavy vehicle speeds are often below 30 km/h, a load duration of 0.5
seconds is used, followed by a rest period of 0.5 seconds. For applications where typical heavy vehicle speeds are expected to be above 40 km/h, a load duration of 0.2 seconds is used, followed by a 0.8 second rest period. The load configuration recommended for different traffic speeds is shown in Table 8.4. For most designs, a load period of 0.2 seconds, followed by a rest period of 0.8 seconds is recommended.

Table 8.4: Proposed Loading Rates for Different Applications

<table>
<thead>
<tr>
<th>Application</th>
<th>Load Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intersections, steep slopes, loading areas with channelised traffic (speed &lt;20 km/h)</td>
<td>0.5 second load 0.5 second rest period</td>
</tr>
<tr>
<td>Normal climbing lanes, medium speed traffic (speed 40 to 80 km/h)</td>
<td>0.2 second load 0.8 second rest period</td>
</tr>
<tr>
<td>High speed traffic (heavy vehicle speed &gt; 80 km/h)</td>
<td>0.1 second load 0.9 second rest period</td>
</tr>
</tbody>
</table>

Once a function relating rut increment to temperature, stress and load repetitions has been derived, it can be used to predict and compare the rut performance of the tested mix for different design situations. Such a fingerprint function can, for example, be used in a computer spreadsheet, in which temperatures and applied loads can be varied to evaluate the sensitivity of the mix to high temperatures and overloading.

The fingerprint function can also be used in a more sophisticated model which predicts seasonal temperature variations and which evaluates traffic loading in a rational manner. One such stochastic analysis model, described in detail in Appendix E, and implemented in the Probabilistic Rut Analysis System (PRORAS), provides a means of performing a sophisticated evaluation of the mix, to provide a reasonable estimate of rut depth development over the design life of the mix.
8.8 Constant Head Permeability Test

Compaction of Briquettes
Combine the aggregates in the correct proportions to meet the final design grading. Add the required amount of binder to the aggregate after heating the ingredients to the correct temperature as defined in Method C2 of TMH1-1986. Mix thoroughly. The mixture is compacted by applying 70 blows with the Marshall hammer, 35 blows to each side of the specimen. Remove the specimen from the mould by means of an extraction jack after cooling sufficiently. Measure the density of the briquette as described in Method C3 and C4 of TMH1-1986.

Preparation for permeability test
Apply a thin layer of silicone sealant around the perimeter of the compacted briquette. Leave to dry, then place in a steel casting mould with 122-125 mm internal diameter and 93 mm high which results in a 12.5 mm wide void around the sample (Figure 8.5). Seal the void between the sample and the inside of the mould using Plaster of Paris taking care not to contaminate the upper and lower surfaces of the briquette with the Plaster of Paris. Allow the Plaster of Paris to dry and apply silicone sealant to the upper and lower exposed edges of Plaster of Paris between the mould and the sample. After the silicone sealant has cured, steel cover plates fitted with an “O” ring to form a watertight seal on the top and bottom edges of the mould, and also fitted with two ball valves, one for water entry and one to act as a bleed valve, are bolted on to the mould. The bottom ball valve of the mould is connected to a constant head permeameter with a water head of 1.0 m and readings of the volume of water coming out of the mould are taken at 5-minute intervals. The test is stopped when the volumes measured for each 5-minute interval are reasonably constant.

![Figure 8.5 Typical Set-Up for the Laboratory Water Permeability Test](image)

The permeability (expressed in / per square metre per hour) of the asphalt sample is determined by using the following formula:

\[
\text{Permeability} = \frac{Q}{A} \quad \text{where } Q = \text{flow in } /h \text{ and } A = \text{area in } \text{m}^2
\]
8.9 Modified Marshall Test

Scope:
This method deals with the determination of the voids content of a cylindrical briquette after each blow of the Marshall hammer on the briquette in order to assess its compaction characteristics.

Apparatus:
Use the method described in the appendix of method C2 of TMH1-1986 for the making of specimens of bituminous mixtures for voids analysis.

The Marshall compactor is equipped with an LVDT fitted on the frame in such a way that the change in height of the briquette can be monitored during compaction. A proximity sensor is used to count the blows applied to the briquette. A photograph of such an arrangement is shown in Figure 8.6. The readings of the LVDT and blows counted by the proximity sensor are electronically carried over to a computer where the data is used for the calculation of voids.

After compaction is completed, the briquette is allowed to cool down and extruded from the mould. Determine the BRD of briquette as described in method C3 of TMH1-1986 and the maximum theoretical relative density as described in method C4 of TMH1-1986 for the same mixture. Calculate the void content of the compacted briquette in accordance with method C3 and determine the final height of the briquette using the volume obtained from the BRD determination and the area of the mould used for compaction of the briquette.

The last reading from the LVDT is taken as the final height of the briquette after 150 blows were applied to the briquette. Subtract 149<sup>th</sup> reading from the final reading and add the difference to the final height reading. Use this as the new height and calculate the volume for this reading. Divide the mass obtained from the briquette after compaction by the volume calculated to determine the void content. Repeat the procedure for all the readings up to blow 80. Skip blows 75 to 80 and adjust the height at blow 75, before the briquette is turned over, to be equal to that at blow 80 and proceed with the calculation for all blows up to blow number 1. Plot the void content against number of blows on a suitable form.

![Figure 8.6: Mounting of the LVDT on the Marshall Compaction Apparatus](image)
9. REFERENCES


13. Based on discussions held between MFC VAN DE VEN (Sabita Chair) and A TAUTE in 1999.


APPENDIX A

NEW TEST METHOD FOR DETERMINATION OF VOLUMETRIC PROPERTIES AND EXPLANATORY NOTES
A1a. Test Method for the Assessment of Aggregate Durability and Soundness (Ethylene Glycol Soundness Test)

Purpose of the test:
To identify the presence of potentially deleterious clay minerals in aggregate used in hot-mix asphalt that may cause the aggregate to swell and to break down.

Procedure:
Sieve a representative sample of aggregate that will be used in the asphalt mix and randomly select 100 pieces of aggregate retained on the largest sieve size. Soak the particles in ethylene glycol for 20 days, after which the number of particles that have broken down in at least two significant parts are counted and expressed as a percentage.

Limits (subject to revision):
No studies have yet been undertaken to establish test limits for aggregates used in hot-mix asphalt. However, based on work conducted on railway roadbeds, a test limit of less than 20 per cent is proposed.

Note: The limits proposed for railway roadbeds are as follows: Ballast < 15%; Base < 30% and Subbase < 60%. Given that aggregates used in hot-mix asphalt are to a fair extent protected against moisture ingress (on account of the bitumen coating), lower requirements than those proposed for railway ballast are recommended. To be validated.

A1b. New Test Method for Determination of Bulk Volume of Compacted Asphalt Briquettes or Cores

Background:
In the past the volume of a compacted asphalt sample was determined by subtracting its mass when suspended in water from its mass when measured in air. The surfaces of asphalt samples, however, are not sealed and there is usually a certain amount of interconnected air voids in such samples. If these pores are large enough they easily fill up with water when the sample is weighed under water. As the main aim is to determine the bulk volume of the samples accurately, it is essential that these voids are not filled with water. In the past, asphalt samples were often coated with wax. However, because the samples are often required for further tests, this practice has generally fallen into disuse. Various techniques have been tried to determine the extent to which these pores get saturated with water. However, none of these has been successful or become widely accepted. The technique proposed here is a refinement of the wrap technique used by the South African National Roads Agency Ltd (SANRAL) on its projects where applicable.

Equipment Needed:
1. Thin soft plastic bags, made of thin, light plastic to ensure that the bag can freely wrap around the contours of an asphalt sample. The bag should be large enough to allow an asphalt sample to be placed into it without damaging the bag. In the test procedure, an asphalt sample is placed inside the bag and the bag and sample are lowered into water. The plastic bag should be large enough to allow the mouth of the bag to be well above the water when the sample is lowered into the water, so that the air around the sample can be expelled freely without water pouring into the bag.
As the bottom seal of the bag may not be watertight, it is recommended that another two seals be applied with an electric bag sealer to ensure that no water leaks into the bag.

2. Small lead or steel balls to weigh down the empty plastic bag when placed in water (the weights remain in the bag with the asphalt sample during volume determination).

3. Thin nylon or wire sling in which asphalt samples can be suspended. It is recommended that a wooden handle be tied to the sling at a length which will ensure that an asphalt sample can be fully lowered into the plastic container without its touching the bottom or sides of the container.

4. Plastic container to hold water and the suspended asphalt sample. The container should be large enough so that the sample inside a plastic bag can be suspended inside without touching the bottom or sides of the container.

5. Electronic scale, capable of measuring up to 16 kg with a precision of 0.1 gram.

6. Retort stands with weights to stabilize the bases of the stands.

7. Horizontal bar to place into retort stands. Approximate length 500 mm.

**Method:**

Note: For speed and accuracy it is imperative that the test be performed by two persons.

1. Preheat the water the water and asphalt samples to 25°C so that no volume corrections are required.

2. Place weighted down retort stands on either side of the scale. Clamp the horizontal bar to the two stands above the plastic container with the water. Tie a handle to the sling at a length which will ensure that the sample is completely covered with water when the asphalt sample is inside the bag.

3. Place the sling and lead weights or balls inside the thin plastic bag and lower the bag and sling into the water container. Lower it until the handle rests on the horizontal bar. Ensure that the bag does not touch the sides or bottom of the container. Zero the scale.

4. Once the scale has been zeroed, the individual samples can be suspended in the sling. While one operator holds the sling and asphalt sample by the handle, the other operator carefully pulls the plastic bag (still with the lead weight or steel balls inside) around the sample. The sample is then carefully lowered into the container, without any water being spilt, until the handle rests on the horizontal bar.

5. Once the bag is at the correct depth and the plastic has folded freely around the sample the mass of the displaced volume of the sample can be read off the scale to the nearest 0.1 g.

6. The procedure is repeated for all the samples. To ensure that as little water as possible is lost during the weighing process, each sample should be removed and replaced directly above the water container.
7. From time to time a plastic bag may get damaged slightly by a sharp protrusion on a sample. If this happens a slight amount of water may leak into the bag. The bag should then be replaced, the scale zeroed with a new plastic bag and the procedure repeated.

8. Where the surface of an asphalt sample is very coarse (i.e. top surface of SMA or porous asphalt) it may be necessary to use some modelling dough to fill the deep surface voids as the plastic bag will not fold into these voids. After the volume of the sample plus dough has been determined, the dough is removed from the sample, rolled into a ball and the displaced volume of the dough ball is determined by weighing it in the plastic bag in water. The mass of the water displaced by the dough ball is then subtracted from the mass of the water displaced by the sample plus dough to give the bulk volume of the asphalt sample.

9. The BRD is determined by dividing the mass (in grams) of the dry sample when weighed in air by the mass of displaced water (in grams). The relative density of water at 25°C is 1.000.
A2. Explanatory Notes for Section 2

Note 1: Sensitivity of Asphalt Tensile Strain and Rutting Parameters to Pavement Structural Variables

The importance of some important pavement structural variables on the asphalt response is illustrated in Figure A1. This figure shows the influence of the immediate support stiffness (i.e. the base stiffness) on the maximum tensile strain at the bottom of the asphalt layer. The basic structure and load variables that were used to derive the strains shown in Figure A1 are shown in Figure A2. The stiffnesses and thicknesses shown in Figure A2 represent the “standard case”. These values were varied to obtain the data points shown in Figure A1. The analysis was performed with a standard layered elastic program.

![Figure A1. Influence of Pavement Structural Variables on Asphalt Response](image)

It is clear from Figure A1 that the base stiffness and the asphalt layer thickness are the most important variables affecting the strain that takes place in the asphalt layer. The base stiffness primarily controls the amount of bending that takes place in the asphalt layer. The asphalt thickness, combined with the load contact area, also determines the relative amount of compression or tension taking place in the asphalt layer. It can also be seen from Figure A1 that the subbase stiffness and asphalt stiffness does not significantly affect the tensile strain in the asphalt layer, especially for base modulus values of more than 200 MPa.
Note 2: Derivation of Suggested Ranges of Base Modulus Values for Thin Asphalt Surfacing

The values shown in Table 2.2 of Section 2 were derived from a stochastic simulation performed with a multilayer elastic analysis program. The pavement structural values assumed for the analysis are shown in Table A1. The load was assumed to be a dual wheel load with a contact pressure of 800 kPa and an applied load per tyre of 20 kN. A spacing between wheels of 350 mm was assumed. The tyre pressure and load properties were assumed to have a 10 per cent coefficient of variation (COV). The stochastic simulation was performed using the Monte Carlo simulation technique with an assumed variation of pavement layer properties as shown in Table A1. All variables were assumed to be normally distributed.

It should be noted that, provided a high enough number of simulations are performed, the Monte Carlo analysis should provide the same average response parameters as would be calculated using a standard layered elastic program with the average material parameters listed in Table A1 as input variables. However, when traffic wander is taken into account, the average stress and strain parameters can be significantly reduced. For this simulation, a standard deviation for traffic wander of 200 mm was assumed. The physical meaning of this standard deviation is illustrated in Figure A3. Figure A4 shows a typical effect of traffic wander on the average maximum tensile strain for the case where no variation exists on other load or pavement variables.

Research has shown that the standard deviation for traffic wander is typically as high as 290 mm for lane widths of 3.7 m. The standard deviation of 200 mm which was adopted to derive typical allowable base stiffness values can therefore be seen as conservative. It should, however, be noted that traffic wander will also be influenced by factors such as rut depth, traffic speed and
typical vehicle widths. For this reason the adoption of a conservative wander parameter seems justified.

Table A1. Pavement Structure used for Derivation of Suggested Base Modulus Values

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average</td>
<td>COV (%)</td>
<td>Average</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>1600</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Granular Base</td>
<td>Varied</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Granular subbase</td>
<td>200</td>
<td>0</td>
<td>0.3</td>
</tr>
<tr>
<td>Subgrade</td>
<td>70</td>
<td>0</td>
<td>0.3</td>
</tr>
<tr>
<td>Deep Support</td>
<td>90000</td>
<td>0</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Apart from the analysis of the effects of traffic wander, the Monte Carlo simulation also provides an indication of the likely distribution of strains due to variations in layer thickness, layer stiffness, load etc. The results of the analysis for different base types are shown in Table A2.

Table A2. Monte Carlo Simulation Results for Thin Asphalt Surfacings

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Maximum Tensile Strain (microstrain) for a Base Modulus of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200 MPa</td>
</tr>
<tr>
<td>50th percentile value</td>
<td>271</td>
</tr>
<tr>
<td>80th percentile value</td>
<td>463</td>
</tr>
<tr>
<td>90th percentile value</td>
<td>516</td>
</tr>
<tr>
<td>95th percentile value</td>
<td>563</td>
</tr>
</tbody>
</table>
Figure A3 Illustration of the Standard Deviation of Traffic Wander

NOTE: Values shown are the average of 1000 simulations

Figure A4. Typical Effect of Traffic Wander on Average Tensile Strain at the Bottom of the Asphalt Layer
Strain values shown in bold in Table A2 were used as the limiting values to determine appropriate support stiffness values for different traffic classes. These values were chosen by simultaneously considering traffic class, expected fatigue life at the limiting strain value, confidence level as well as experience with backcalculated support stiffness values for different classes of pavement structures.

For asphalt thicknesses of 40 mm and less, the base stiffness is the primary structural variable that influences tensile strain at the bottom of the asphalt layer (see Note 1 above). The values shown in Table A2 can therefore be assumed to also apply to structures for which the asphalt, subbase and subgrade stiffnesses differs somewhat from that shown in Table A1. The values shown in Table A2 will, however, be very conservative for thin (30 mm or less) asphalt layers. However, owing to the lack of experience of the structural performance of such thin layers in South Africa, it is recommended that the suggested ranges of base moduli shown in Table 2.2 of Section 2 be adhered to until more performance data for such thin asphalt layers become available.

It should be noted that the values shown in Table A2 were based on the assumption of a static load. For traffic speeds of more than about 40 km/h the tensile strain can be reduced significantly (up to 50 per cent) due to dynamic effects. Designers should however, be aware that the fatigue resistance is somewhat reduced at higher loading rates, a factor which may partially offset the reduction in tensile strain which is caused by dynamic effects.

**Note 3: Derivation of Suggested Ranges of Base Modulus Values for Asphalt Bases**

The suggested ranges of subbase support values for thicker (> 80 mm) asphalt bases were derived in the same way as that for thin asphalt surfacings (see Note 2 above), except that the pavement structure was changed to reflect a thicker asphalt base under a thin asphalt surfacing. Load parameters were identical to that used for the thin asphalt surfacings. Table A3 shows the layer stiffnesses and thicknesses assumed for the analysis.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average</td>
<td>COV (%)</td>
<td>Average</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>1600</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Granular Base</td>
<td>1600</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Subbase</td>
<td>Varied</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Subgrade</td>
<td>200</td>
<td>0</td>
<td>0.3</td>
</tr>
<tr>
<td>Deep Support</td>
<td>90000</td>
<td>0</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The calculated strains at the bottom of the asphalt base is summarized in Table A4. It will be noted that the stiffness for the asphalt base and surfacing shown in Table A4 are at the lower end of the range of typical stiffness values for these materials. As noted in the previous section, the asphalt stiffness does not significantly affect the strain at the bottom of the thin surfacing. However, In the case of the thick asphalt base, the stiffness may have a greater effect on tensile strain at the bottom of the base.
The values shown in Table A2 can therefore be assumed to be conservative as far as material stiffness is concerned. The results are also conservative as far as the dynamic effects are concerned. Despite these observations, it is felt that the suggested subbase stiffness values shown in Table 2.3 of Section 2 are at the lower end of what may be expected for the high quality pavement structures which are normally associated with asphalt bases, as recommended in TRH4². Pavement design considerations may therefore to some extent override the suggested minimum support stiffness values shown in Table 2.3.

Table A4. Monte Carlo Simulation Results for Asphalt Bases

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Maximum Tensile Strain (microstrain) for a Subbase Modulus of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>300 MPa</td>
</tr>
<tr>
<td>50th percentile value</td>
<td>159</td>
</tr>
<tr>
<td>80th percentile value</td>
<td>190</td>
</tr>
<tr>
<td>90th percentile value</td>
<td>206</td>
</tr>
<tr>
<td>95th percentile value</td>
<td>218</td>
</tr>
</tbody>
</table>
APPENDIX B
BASIC PRINCIPLES OF SPATIAL COMPOSITION
B1. Approach and Basic Principles

Most of the recently developed approaches to mix design include a consideration of volumetric or spatial design concepts. In fact, modern approaches to mix design often rely entirely on volumetric design principles to determine the correct proportions of coarse aggregate, fine aggregate, filler as well as optimum binder content. Although the volumetric design is often labelled as “Level 1” (as for example in the Superpave\textsuperscript{15} and Australian design methods\textsuperscript{25}), higher design levels simply involve more complex forms of testing and evaluation which primarily serve as validation and to increase confidence in the performance of the mix. Volumetric principles therefore form the basis of a modern approach to HMA design, and designers should therefore be familiar with the most important facets of volumetric design. These facets relate to the packing patterns of aggregates and the determination and evaluation of relative volumes of different components.

The concept of packing mechanism is central to an understanding of spatial design. Prior to the selection of a target gradation and the calculation of volumetric design parameters, designers should be aware of the intended spatial composition of the planned mix. In particular, designers should be aware of the packing characteristics of the planned mix type and how these influence the volumetric design parameters.

The type of skeleton structure that is aimed for in the design should be kept in mind, and the evaluation and selection of the gradation should ensure that the appropriate packing mechanism is attained. This concept is shown graphically in Figure B1.

Two opposing packing mechanisms govern the packing of aggregates:
- **Substitution**, in which the space occupied by the fine aggregate fraction is replaced through an increase in the concentration of the coarse aggregate fractions. This mechanism applies to sand skeleton mixes.
- **Filling**, in which the spaces between coarse aggregates are filled by an increase in the concentration of fine aggregate. This mechanism applies to stone skeleton mixes.

These two packing mechanisms serve different purposes and have different advantages and disadvantages as far as stability, durability and compactibility are concerned. The selection of a target gradation and analysis of volumetric parameters should be relevant for the particular type of packing mechanism that is aimed for in the design. Figure B2 shows the volumes that are involved in a packing of binder-coated aggregates. These volumes are:

i) Volume of solid aggregate;
ii) Volume of cavities in the aggregate which cannot be penetrated by binder;
iii) Volume of binder absorbed into cavities in the aggregate (absorbed binder);
iv) Volume of binder not absorbed (effective binder), and
v) Volume of voids between coated particles (air voids).

These five volumes are the building blocks which an HMA designer should balance and optimize to ensure that the mix meets the required performance criteria. To simplify this balancing act, some of these volumes are combined to form a few essential volumetric properties which are known to relate to mix performance. Of these, the most basic, and which serves as the cornerstone of volumetric design, is the quantity known as the **voids in the mineral aggregate (VMA)**.
Figure B1. Classification of Mixes Based on Skeleton Type

Figure B2. Schematic Illustration of Volumes in a Binder Aggregate System
VMA is defined as the volume of voids between coated particles plus the volume of effective binder. The VMA is therefore the volume that is available for filling with binder, plus any inter-particle voids which may be unfilled after the binder has been added.

VMA thus includes volumes (iv) and (v) defined above, and excludes volumes (i) to (iii). If any of the volumes (i) to (iii) decreases, the VMA will increase, and vice versa. Consideration of volumes (i) to (iii) will therefore show that VMA is affected primarily by gradation and aggregate characteristics. Both of these relate to the packing characteristics of the aggregates. VMA is also affected by the compaction effort, which determines the degree of packing for a given gradation and aggregate type. Table B1 summarizes the individual elements that affect VMA.

It is possible to select a combination of fine and coarse aggregate and a gradation which will result in the lowest possible VMA. However, such gradings may have too little VMA to accommodate sufficient binder and still have enough air voids between the coated particles to meet stability requirements. VMA is therefore partly determined by the type of gradation as well as the maximum aggregate size. These considerations are in turn guided by the mix selection guidelines provided in Chapter 2.

### Table B1. Relationship between Mix and Construction Parameters and VMA

<table>
<thead>
<tr>
<th>Property</th>
<th>General relationship with VMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle size and distribution</td>
<td>Complex, but denser gradations generally lead to decreased VMA.</td>
</tr>
<tr>
<td>Maximum aggregate size</td>
<td>Larger aggregates reduce VMA</td>
</tr>
<tr>
<td>Aggregate shape</td>
<td>Higher angularity increases VMA</td>
</tr>
<tr>
<td>Aggregate texture</td>
<td>Rougher textures increase VMA</td>
</tr>
<tr>
<td>Aggregate rugosity (geometric irregularity)</td>
<td>Greater irregularity increases VMA</td>
</tr>
<tr>
<td>Filler content and type</td>
<td>Extremely fine particles (&lt; 10 microns) may function as a binder extender, causing the available VMA to decrease.</td>
</tr>
<tr>
<td>Absorption potential</td>
<td>For a given binder content, higher absorption will lead to increased VMA</td>
</tr>
<tr>
<td>Layer thickness</td>
<td>Lower layer thickness generally leads to higher surface area to volume ratio. This causes the measured VMA to increase.</td>
</tr>
<tr>
<td>Compaction effort</td>
<td>More compaction (including compaction by traffic) will lead to reduced VMA</td>
</tr>
</tbody>
</table>

It was noted earlier that VMA comprises the voids filled with effective (unabsorbed) binder, plus the voids between coated particles. For a given mix type and traffic class, the required void contents between coated particles are determined primarily through experience. For most mixes it is required that the void contents should lie between 3 and 6 per cent. The voids requirement ensures that stability (rutting resistance) requirements are met. The specified void contents are required to allow for thermal expansion of the binder during hot weather, and also to accommodate further reductions in VMA resulting from long-term traffic compaction.
In addition to a minimum voids content, the volumetric design requires that the VMA be sufficiently high to allow enough binder into to mix to ensure that stability and workability requirements are met. Thus:

\[
\text{Optimum VMA} = \frac{\text{Volume of effective binder required for workability/durability}}{\text{Volume of binder required for stability}} + \frac{\text{Volume of voids required for stability}}{\text{Volume of binder required for stability}}.
\]

Although the above relationship seems simple, it requires some experience to truly optimize the VMA requirements. The difficulty lies in the selection of an appropriate binder content to satisfy workability and durability requirements. Durability and workability will improve as the binder content increases. However, a point may be reached where the binder content is so high that it actually forces the aggregates apart, thereby “artificially” increasing the VMA and destroying the skeleton of stone-to-stone contacts. Figure B3 illustrates this situation.

Figure B3 shows the typical shape of a VMA curve. It is somewhat parabolic in shape with a minimum VMA point. For binder contents to the left of this minimum, the VMA will decrease as the binder content increases because of the lubrication afforded by the binder, which leads to closer packing and thus decreased VMA. For binder contents to the right of the minimum VMA (i.e. above point “A” in Figure B3), the VMA is increased by “invasion” of the aggregate structure. This means that the aggregates are starting to float in the binder - a situation which may lead to severe loss in stability.

![Diagram](image_url)

Figure B3. Relationship between Binder Content, Voids and VMA

Another important element of VMA determination is the level of compaction used during the design. Figure B4 illustrates the influence of compaction effort or densification on VMA. Typically, both the minimum VMA and the binder content required to achieve minimum VMA decrease with increasing compaction effort. Figure B4 also shows the importance of choosing an appropriate compaction level during the design. If compaction effort A is used to determine the design binder content and the actual densification due to construction and traffic is equal to compaction effort B, then the selected binder content will eventually be too high. This is likely to result in an
overfilling of available void space. This emphasizes the importance of choosing a compaction device which accurately simulates traffic compaction, and of selecting a compaction level that is appropriate for the traffic class.

All mix design methods involve trial mixes made with different binder contents. At each binder content, VMA, voids, stabilities, densities, etc., are measured and plotted versus binder content. The designer then has to optimize all these properties through the selection of an optimum binder content. There are different ways in which the selection of an optimum binder content can be approached.

The conventional Marshall design approach is to determine the average optimum binder content based on maximum stability (Marshall stability), maximum density and specified air voids content. These maximum values as well as the Marshall flow and VMA at the selected binder content are then compared with specified criteria. Failure to meet any of these criteria requires a redesign of the mix. This may involve a gradation change, addition or reduction of filler or a change in aggregate type. It can be seen that the success of this method is largely dependent on the validity of the stability and flow test as well as the validity of the specified criteria.

The modern volumetric design approach is somewhat different. In this approach, the design binder content is based primarily on the specified voids content. The VMA at the design binder content is then evaluated to ensure that the binder content is not so high that it forces open the aggregate structure. In essence, this means that the design binder content should be close to, or to the left of, the minimum VMA point on a plot of VMA versus binder content. A re-examination of the definition of VMA will show that the simultaneous evaluation of voids and VMA effectively means that the voids filled with binder (VFB) are evaluated.
The evaluation of voids and VMA should ensure that the mix has sufficient resistance to rutting, provided that the compaction level used in the mix design process is high enough to simulate densification under traffic. The second evaluation pertains to durability and is aimed at ensuring that the binder content is high enough to provide an adequate coating (or film thickness) around the aggregates. If the stability or durability/workability requirements are not met, then the packing characteristics of the mix (and thus the VMA) should be adjusted. Table B1 suggests which mix parameters could be changed to alter the packing and VMA characteristics.

The two paragraphs above summarize the essence of the volumetric design approach. The design process is simple, and is basically centred around the selection of a binder content to provide a specified void content (normally 3 to 5 per cent). This void content is based on overwhelming evidence that suggests that mixes with void contents below 3 per cent after compaction are prone to rutting.

Despite the apparent simplicity of the volumetric design approach, mixes with widely varying properties can be designed using the same voids criteria. This follows from the dependence of the VMA on the packing characteristics of aggregates. A finer gradation with smoother, rounder aggregate would have higher VMA and thus would be able to accommodate more binder at the specified voids content. Although such a mix would not be as stable as a large stone mix with rough angular aggregates, it would be much more durable and easier to construct, and thus would be more suited to low-volume applications. Minimum VMA criteria for continuously graded mixes are shown in Table B2\textsuperscript{27}.

Table B2. Minimum Percent Voids in Mineral Aggregate (VMA)\textsuperscript{27}

<table>
<thead>
<tr>
<th>Nominal Max. Particle Size (mm)\textsuperscript{1}</th>
<th>Minimum VMA for Design Air Voids of\textsuperscript{2}</th>
<th>3.0%</th>
<th>4.0%</th>
<th>5.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.18</td>
<td>21.5</td>
<td>22.5</td>
<td>23.5</td>
<td></td>
</tr>
<tr>
<td>2.36</td>
<td>19.0</td>
<td>20.0</td>
<td>21.0</td>
<td></td>
</tr>
<tr>
<td>4.75</td>
<td>16.0</td>
<td>17.0</td>
<td>18.0</td>
<td></td>
</tr>
<tr>
<td>9.5</td>
<td>14.0</td>
<td>15.0</td>
<td>16.0</td>
<td></td>
</tr>
<tr>
<td>12.5</td>
<td>13.0</td>
<td>14.0</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>19.0</td>
<td>12.0</td>
<td>13.0</td>
<td>14.0</td>
<td></td>
</tr>
<tr>
<td>25.0</td>
<td>11.0</td>
<td>12.0</td>
<td>13.0</td>
<td></td>
</tr>
<tr>
<td>37.5</td>
<td>10.0</td>
<td>11.0</td>
<td>12.0</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>9.5</td>
<td>10.5</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>63</td>
<td>9.0</td>
<td>10.0</td>
<td>11.0</td>
<td></td>
</tr>
</tbody>
</table>

\textsuperscript{1} The nominal maximum particle size is one size larger than the first sieve to retain more than 10 per cent

\textsuperscript{2} Interpolate minimum voids in the mineral aggregate (VMA) for design air void values between those listed
From the above, it can be seen that volumetric design centres around the following concepts:

i) packing characteristics of the chosen aggregate size and gradation as well as the resulting VMA (determined by selection of mix type, gradation and maximum aggregate size);

ii) selection of an appropriate compaction level to simulate densification resulting from construction and traffic (determined by traffic level);

iii) selection of a binder content to ensure that voids are within the desired range, and

iv) evaluation of durability and workability requirements.

Chapter 4 of these guidelines provides a framework for achieving an optimal balance between the abovementioned elements. Criteria for selection of compaction level, voids content and evaluation of VMA and VFB are given in Chapter 4, as well as in the sections dealing with specific mix types.

**B2. Validation of Volumetric Design**

In principle, a design which has a sound volumetric basis meeting all the criteria set out in Chapter 4, and for which the components were properly evaluated according to the criteria provided in Chapter 3, should perform as expected (one should perhaps say - “as designed”). However, because of the complexity of binder-filler-aggregate systems, and because of the many elements that need to be simultaneously considered, the design mix is normally also subjected to physical tests. These tests are primarily aimed at increasing confidence that the mix will meet the minimum expected performance criteria.

The level of confidence that is required during a mix design is primarily determined by the traffic level. The consequences of failure on a high volume road are such that more complex and expensive performance tests are justified. For low volume road applications, the designer may feel comfortable in using less well validated (but also less expensive) tests to evaluate performance. This is the reason for having more than one design level. It should be noted, however, that the only difference between the different design levels is the performance tests that are required to enhance confidence in the expected performance. The basic design method, however, is the same for all design levels, and centres around the volumetric approach described above.

When validating the volumetric design, the designer should be acquainted with the differences between stone- and sand-skeleton mixes in terms of their packing characteristics, workability and density specifications. Stone-skeleton mixes are generally considered to have good workability and can fairly easily be compacted to design (refusal) void contents, assuming that at the design (refusal) void content, point-to-point contact has been achieved. These considerations should be taken into account when specifying field densities and evaluating such mixes in a laboratory environment.

Case studies have shown that if the assessment of the resistance to permanent deformation of stone-skeleton mixes is not conducted at the design (refusal) void content, their performance may be poor in any form of rutting simulation test. On the other hand, optimum resistance to permanent deformation of well-designed stone-skeleton mixes is achieved at refusal density, provided that the voids in the coarse aggregate (VCA) are not overfilled.
The above may suggest that current density specifications used for sand-skeleton mixes (i.e. (97% minus voids) times the maximum theoretical relative density) may not be equally applicable to stone-skeleton mixes as these could lead to permeable and/or low rut-resistant mixes. Hence, 95-96 per cent of the maximum theoretical relative density could possibly be recommended, provided that the optimum spatial composition (including the binder) is selected at refusal density.

B3. Measurement and Calculation of Volumetric Properties

To facilitate volumetric design and evaluation, the following quantities need to be measured:

i) Bulk relative density (BRD) of the various aggregate fractions, as well as the combined bulk relative density of the combined aggregate;

ii) Theoretical maximum relative density of the asphalt mix;

iii) Bulk relative density of the compacted asphalt mix;

iv) Effective relative density of combined aggregate;

v) Effective asphalt content;

vi) Specific density of the binder, and

vii) Absorption potential of aggregate;

These quantities will facilitate computation of VMA, voids content and VFB. Formulas for calculation of these quantities are provided in Section 5.

B4. SUPERPAVE 2000: Improved Standards for a New Millennium

Based on continued research in the US, expert opinion and implementation experience, modification were made to the original Superpave procedures and recommendations for the volumetric design of HMA. These were intended to clarify and simplify the mix design process and improving the resulting products.

Appended to Appendix B is Technical Brief #17 produced by the Canadian Strategic Highway Research Program (C-SHRP) in which the latest improvements and updates are presented and discussed. These relate to the following AASHTO standards:

- MP2: Specifications for Superpave Volumetric Mix design;
- PP2: Practice for Mixture Conditioning of HMA;
- PP28: Practice for Performing Superpave Volumetric Designs for HMA;
- TP4: Method for Preparation and Determining the Density of HMA by Means of the SHRP Gyratory Compactor
SUPERPAVE 2000 – Improved Standards for a New Millennium

September 1999

Unveiled in 1992, the Superpave system represented a fundamentally new system for designing asphalt concrete mixtures. The “performance-based” nature of the system not only promoted improved pavement life, but also the potential ability to predict pavement performance based on accelerated testing. Implementation of Superpave was slow at first, reflecting the industry’s hesitation to implement a new and unproven technology.

In 1996, the American Association of State Highway and Transportation Officials (AASHTO) Task Force on SHRP Implementation formed the Superpave Lead State Team to aid the uniform implementation of Superpave in the United States. The goals of the Lead State Team were to document and share experiences, further development and provide guidance related to the practical application of the Superpave technology.

As with any new system, certain challenges, or “growing pains”, have been observed during Superpave implementation. Such challenges were expected since the original Superpave system was not released as a finished product, but more of a solid foundation for continued improvement. Continued research coupled with State and Provincial field implementation experience have provided the necessary information to upgrade the Superpave system to a more powerful tool for government agencies, contractors and consultants and the general public.

BACKGROUND

Under the auspices of AASHTO, national standards for the Superpave system are maintained and published through the Subcommittee on Materials (SOM) representing all 50 US states. In August of 1998, the SOM Technical Section for Asphalt Mixtures established a five-state Task Force to consider, prepare and recommend changes to the Superpave mix design system for balloting by the entire SOM. Many of the Task Force members were members of either the Superpave Lead State Team or the Mix Expert Task Group (ETG) or both.

Changes to the Superpave system were proposed to the Task Force from the following four main sources:

• NCHRP Project 9-9: “Refinement of the Superpave Gyratory Compaction Procedure”;

• Superpave Lead State Guidance;

SUPERPAVE™ is the product of the asphalt research undertaken as part of the Strategic Highway Research Program (SHRP) and it integrates performance based specifications, test methods, equipment, testing protocols, and a mixture design system. SHRP was established by the United States Congress in 1987 as a five-year, $150 million research program to improve the performance and durability of highways and to make them safer for motorists and highway workers. As one of four technical programs, the asphalt program received $850 million for the development of performance based asphalt specifications to directly relate laboratory analysis with field performance. As a follow-on program to SHRP Congress established in the Intermodal Surfaced Transportation Efficiency Act of 1991 programs to implement SHRP products and to continue SHRP’s Long-Term Pavement Performance (LTPP) program. The Canadian Strategic Highway Research Program (C-SHRP) is directed at extracting the benefits of the US work for Canada.
• Input from the Mix ETG;
• Suggestions by member States

The Task Force reviewed the recommended changes and proposed revisions to four Superpave standards. A ballot was prepared and sent to all SOM members for voting by February 6, 1999. The results were officially published in May of 1999. Improvements and updates have been made to the following AASHTO standards:

• MP2: Specifications for Superpave Volumetric Mix Design;
• PP2: Practice for Mixture Conditioning of Hot Mix Asphalt (HMA);
• PP28: Practice for Designing Superpave Volumetric Design for HMA
• TP4: Method for Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specifications by Means of the SHRP Gyratory Compactor

THE NEW STANDARDS

Based on continued research, expert opinion and implementation experience, the standard revisions are intended to clarify and simplify the mix design process and improve the resulting product. More detailed description of the changes are presented in the following nine sections.

1. Improved and Simplified \( N_{\text{design}} \) Table

The original Superpave \( N_{\text{design}} \) table was constructed around a range in traffic of 10 to 20 million ESALs. This traffic range was assigned an \( N_{\text{design}} \) value of 100 with other values estimated based on higher or lower traffic volumes. The improved \( N_{\text{design}} \) table, shown in Table 1, is based on research performed during NCHRP 9-9 by the National Centre for Asphalt Technology (NCAT) and the \( N_{\text{design}} \) II experiment completed by the Asphalt Institute.

...Reduction in Number of Design Traffic Levels

As shown in Table 1, a number of significant changes have been made to the \( N_{\text{design}} \) table. The first and most obvious change is the reduction in the number of design traffic levels from seven to four (five levels are used for materials selection). The central \( N_{\text{design}} \) value of 100 was assigned for the 3 to less-than 30 million ESAL traffic range, however, other values are now based on sensitivity analysis of volumetric properties and mixture stiffness.

...Consolidation of Temperature Columns

Another significant change to the \( N_{\text{design}} \) table was the consolidation of the four temperature columns into a single column, reflecting two main observations. First, it was observed that volumetric properties displayed low sensitivity to the number of gyrations between temperature columns. Second, the original \( N_{\text{design}} \) research did not recognize that stiffer binders are used in hotter climates, thereby making the need for separate temperature columns redundant.

...Mix Design for 20-Year ESALs

The design ESAL ranges shown in Table 1 represent the cumulative number of ESALs for a 20-year period. Although some pavements may not be designed for a 20-year life, the compaction level must be selected based on cumulative ESAL count over 20 years to account for the effects of ESAL application rate. Experience has shown that rutting damage can occur within the first few years of a pavement's life, therefore, the rate of loading must be considered. For example, a pavement having a 5-year intended life might experience 0.58 million ESALs per year over the 5-year period for a total of 2.9 million ESALs. However, the mixture must be designed based on the 20 year estimated life of 11.6 million ESALs (0.58 million ESALs per year multiplied by 20 years) so that mix may better resist the effects of premature rutting. Traffic growth over the 20 years need not be considered when inflating the intended ESAL count.

...Consideration of Location in Pavement Structure

To account for the effects of pavement thickness, NCHRP 9-9 research recommended that the required design compaction should be lowered by one level for mixtures that are located more than 100 mm below the pavement surface. This practice should not affect the structural strength of the pavement since mixtures at that depth do not “feel” the same stresses as those imposed at the pavement surface. It is anticipated that this practice will result in underlyng mixtures with higher asphalt contents, thereby improving durability and moisture resistance.
However, a major note to this change is the consideration of construction schedule during mix design. A mixture designed at a lower-than-required compaction level will likely sustain premature deterioration if subjected to traffic for an extended period of time prior to overlay.

For practical purposes, if less than 25% of an underlying layer is within 100 mm of the pavement surface, the layer should be considered to be below 100 mm for mix design purposes. For example, a 100-mm (4 in.) binder course may be designed one compaction level lower than normal if it is located between 75mm to 100mm (3 to 4 in.) below the surface of the wearing course. However, if the binder coarse is only 75mm in thickness, it must be designed at the standard compaction level since more than 25% of its thickness is within 100mm from the surface of the wearing coarse. This 25% rule also applies to Superpave greater-than 100mm aggregate consensus properties.

2. New Mixture Conditioning Procedures

The long term ageing procedure specified in PP-2 remains unchanged at 120 ± 0.5 hours at 85 ± 3°C for compacted specimens. An overall change to the specification calls for the universal replacement of the term “ageing” with “conditioning.”

The former standard called for a single procedure for short-term asphalt mixture ageing for both volumetric mix design and mechanical property testing. That process subjected loose asphalt mixtures to conditioning in a force draft oven for 4 hours at 135°C prior to either volumetric analysis or mechanical testing.

<table>
<thead>
<tr>
<th>Design ESALs(1) (Million)</th>
<th>Gyralatory Compaction Parameters</th>
<th>Typical Roadway Description(2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$N_{initial}$</td>
<td>$N_{design}$</td>
</tr>
<tr>
<td>&lt; 0.3</td>
<td>6</td>
<td>50</td>
</tr>
<tr>
<td>0.3 to &lt; 3</td>
<td>7</td>
<td>75</td>
</tr>
<tr>
<td>3 to &lt; 30</td>
<td>8</td>
<td>100</td>
</tr>
<tr>
<td>$\geq$ 30</td>
<td>9</td>
<td>125</td>
</tr>
</tbody>
</table>

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years and choose the appropriate $N_{design}$ level.

(2) Typical road applications as defined by A Policy on Geometric Design of Highways and Streets, 1994, AASHTO.

Table 1: New Superpave $N_{design}$ Table (from AASHTO PP-28)
... New Short Term Conditioning Procedure for Mix Design

Based on results from NCHRP 9-9, NCAT concluded that there is no practical difference in mixture volumetric properties when either 2 or 4 hours of short term conditioning is performed. Therefore, the new PP-2 standard allows a 2-hour conditioning procedure for volumetric mixture design that will result in faster mixture design development. PP-2 also specifies that short term conditioning for volumetric mixture design is to be completed at the mixture’s compaction temperature. This represents a change from the 135°C conditioning temperature previously specified. The procedure for short term conditioning of specimens for mechanical testing remains at 4 hours at 135°C based on FHWA research.

...Increased Capacity of Force Draft Ovens

The second item has specified an increase in the specified capability of the force draft oven to handle modified asphalt binders. PP-2 now calls for a force draft oven thermostatically controlled from room temperature to 176 ± 3°C (up from 150 ± 3°C). This change is required to address the change to mixture conditioning at the mixture’s compaction temperature, as noted above.

3. Allowance for New Types of Gyratory Compactors

Most of the changes to standard TP-4 were made to ensure consistency between the standards PP-2 and PP-28. However, a significant change made to TP-4 involved an update of the references to Superpave gyratory compactors and compaction procedures. Specifically, the references were made more generic to allow the use of the various compactors commercially available that currently meet the Protocol for Evaluation of Superpave Gyratory Compactors.

4. Specimen Preparation with Gyratory Compactor

The previous gyratory compaction procedures specified by PP-28 required that specimens be compacted to $N_{\text{max}}$ with densities and volumetric properties then backcalculated to $N_{\text{design}}$. This practice can cause errors in the calculated values at $N_{\text{design}}$. The new PP-28 standard specifies that specimens for the volumetric mix design procedure should be compacted to $N_{\text{design}}$, not $N_{\text{max}}$, since the mixture design is based on the volumetric properties at $N_{\text{design}}$.

Additional specimens must now be compacted to $N_{\text{max}}$ only for verification that the mixture does not exceed the maximum density of 98%.

<table>
<thead>
<tr>
<th>Design ESALs$^{(1)}$ (Million)</th>
<th>Coarse Aggregate Angularity Minimum Percent</th>
<th>Uncompacted Void Content of Fine Aggregate Angularity (Minimum Percent)</th>
<th>Sand Equivalent Minimum Percent</th>
<th>Flat and Elongated $^{(2)}$ Maximum Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 100 mm</td>
<td>&gt; 100 mm</td>
<td>&lt; 100 mm</td>
<td>&gt; 100 mm</td>
</tr>
<tr>
<td>&lt; 0.3</td>
<td>55/-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.3 to &lt; 3</td>
<td>75/-</td>
<td>50/-</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>3 to &lt; 10</td>
<td>85/80$^{(2)}$</td>
<td>60/-</td>
<td>45</td>
<td>40</td>
</tr>
<tr>
<td>10 to &lt; 30</td>
<td>95/90</td>
<td>80/75</td>
<td>45</td>
<td>40</td>
</tr>
<tr>
<td>≥ 30</td>
<td>100/100</td>
<td>100/100</td>
<td>45</td>
<td>45</td>
</tr>
</tbody>
</table>

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years and choose the appropriate $N_{\text{design}}$ level.

(2) “80/85” denotes that 85% of the coarse aggregate has one fractured face and 80% has two or more fractured faces.

(3) Criterion based upon a 5:1 maximum to minimum ratio.

Table 2: Consolidated Aggregate Consensus Property Table for Superpave (from MP-2)
5. Consolidation of Aggregate Consensus Property Tables

While the original MP-2 standard contained three separate tables outlining specifications for aggregate consensus properties, the new MP-2 has consolidated the tables for coarse aggregate angularity, fine aggregate angularity and sand equivalent criteria into a single table as replicated in Table 2. Unlike the new N<sub>design</sub> table that uses four design traffic levels, five levels are used to determine the aggregate consensus properties. The extra traffic level is the result of splitting the 3.0 to less than 30.0 million ESAL level into a 3.0 to less than 10 million ESAL range and a 10 to less than 30 million ESAL range.

6. Consolidation of Superpave Volumetric Mix Design Tables

To further improve the clarity and ease of use of the Superpave system, the original design tables concerning Voids in the Mineral Aggregate (VMA) Criteria, Voids Filled with Asphalt (VFA) Criteria and Density Requirements for Mix Design have been consolidated into a single table as replicated in Table 3. Numerous revisions have been made within these new consolidated tables as presented in the following sections.

...Superpave for Low-Volume Roads

The previous N<sub>initial</sub> criteria prevented the design of (many) mixtures with gradations passing above the restricted zone. This severely limited the applicability of Superpave for low-volume roads since fine-graded mixtures or mixtures without manufactured sands typically display high density at N<sub>initial</sub>, usually exceeding the maximum value of 89.0%.

Realising that mixtures placed on low-volume roads do not typically need the same structural strength as required for high ESAL environments, the values of N<sub>initial</sub> for the two lowest traffic levels have been increased to 91.5% and 90.5%, respectively as shown in Table 3. These fine graded Superpave mixtures have surface textures similar to conventional mixtures and may be more easily compacted than coarse-graded Superpave mixtures of the same design level.

<table>
<thead>
<tr>
<th>Design ESALs&lt;sup&gt;(1)&lt;/sup&gt; (Millions)</th>
<th>Required Density (Percent of G&lt;sub&gt;mm&lt;/sub&gt;)</th>
<th>Voids in the Mineral Aggregate (VMA) (Minimum Percent)</th>
<th>Nominal Maximum Aggregate Size (mm)</th>
<th>Voids Filled with Asphalt (VFA) (Minimum percent)</th>
<th>Dust-to-Binder Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N&lt;sub&gt;initial&lt;/sub&gt;</td>
<td>N&lt;sub&gt;design&lt;/sub&gt;</td>
<td>N&lt;sub&gt;max&lt;/sub&gt;</td>
<td>37.5</td>
<td>25.0</td>
</tr>
<tr>
<td>&lt; 0.3</td>
<td>≤ 91.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3 to &lt; 3</td>
<td>≤ 90.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 to &lt; 10</td>
<td>96.0</td>
<td>&lt; 98.0</td>
<td>12.0</td>
<td>13.0</td>
<td>14.0</td>
</tr>
<tr>
<td>10 to &lt; 30</td>
<td>≤ 89.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years and choose the appropriate N<sub>design</sub> level.

(2) For 9.5mm nominal maximum size mixtures the specified VFA range shall be 73% to 76% for design traffic levels ≥ 3 million ESALs.

(3) For 25.0mm nominal maximum aggregate size mixtures the specified lower limit of the VFA shall be 66% for design traffic levels < 0.3 million ESALs.

(4) For 37.5mm nominal maximum aggregate size mixtures the specified lower limit of the VFA shall be 63% for all design traffic levels.

Table 3: Superpave Volumetric Mixture Design Requirements (adapted from MP-2)
...Updated VFA and VMA Ranges

The original limiting values of VFA produced narrow ranges for VMA, increasing the difficulty for asphalt mix designers to meet the Superpave specifications. For example, the limiting VFA for a 9.5 mm mixture designed for traffic level greater than 3 million ESALs was originally 75.0%, thereby requiring VMA to fall between 15.0 to 16.0% to meet the specification. The new MP-2 standard has raised the upper limit for VFA for 9.5 mm mixtures designed for greater than 3 million ESALs to 76%, providing a larger range of acceptable VMA between 15.0 to 16.7%. Similar changes to VFA have been made for other size mixtures as outlined in Table 3.

...Increased Dust-to-Binder Ratio

To ensure sufficient asphalt-aggregate bond, the dust-to-binder ratio was specified by some agencies as part of their volumetric mix design criteria. The most widely accepted range was 0.6 to 1.2 based on total binder content. The previous Superpave standards included this same range, however, the dust-to-binder ratio was calculated based on effective asphalt content. This practice produced higher dust-to-binder ratios compared to those based on total asphalt content. To counter this effect, Superpave now specifies that dust-to-binder ratios from 0.6 to 1.6 are acceptable based on positive experience with these mixtures.

7. New Binder Selection Adjustments for Traffic Level and Speed

Superpave binder selection is initially completed using the high and low temperature algorithms based on local temperature data. However, adjustments are then made to the high temperature grade according to traffic level and speed to accommodate the effects of load rate and volume toward premature pavement rutting. The original Superpave standards addressed either slow traffic speed or high traffic volume, but not both. The new binder adjustment table shown in Table 4 displays the adjustments required for various combinations of traffic speed and volume.

8. New Low Temperature Calculation for Binder Selection

Based largely on C-SHRP research, the original Superpave low-temperature algorithm was deemed overly conservative for selecting low temperature binder grades. Subsequently, both the US-LTPP program and the Transportation Association of Canada (TAC) have developed new low temperature algorithms based on weather station data from Canada.
and the United States. In October 1998, a review of both the US-LTPP and TAC algorithms was prepared as C-SHRP Technical Brief # 15 – “Revised Low Temperature Algorithms for Superpave Mix Design System.” This technical brief is available through the C-SHRP website at www.tac-atc.ca/programs/cshrp.htm.

The new US-LTPP low temperature algorithm has been incorporated into a new software package called LTTPBind, replacing the original SHRPBind program. This software program may be acquired at no cost through the Turner-Fairbank Highway Research Centre website at www tfhrc.com.

**RAP - THE MISSING LINK**

A major component unaddressed by the AASHTO ballot was the inclusion of reclaimed (or recycled) asphalt pavement (RAP) into asphalt mixtures. Using RAP in Superpave is not only advantageous with respect to costs, but is also environmentally friendly. The AASHTO SOM was not able to include RAP into the ballot, as the results of NCHRP 9-12 were not available at that time. The NCHRP 9-12 project commenced in April of 1997 at the North Central Superpave Centre at Purdue University to address issues surrounding the incorporation of RAP in Superpave mixtures. Completion of this study is anticipated in mid 1999.

In the interim, the Superpave Lead State Team recommended the adoption of the report entitled “Guidelines for the Design of Superpave Mixtures Containing Reclaimed Asphalt Pavement (RAP)” prepared by John Bukowski, chair of the Superpave Mixtures ETG. The guideline provides a three-tiered approach to RAP based on the proportion of RAP added to the asphalt mixture as follows:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>At agency discretion</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>No change</td>
</tr>
<tr>
<td>2</td>
<td>Yes</td>
<td>Yes</td>
<td>No, unless blending chart used</td>
<td>Yes</td>
<td>One grade lower or use blending chart</td>
</tr>
<tr>
<td>3</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Use blending chart</td>
</tr>
</tbody>
</table>

**Tier 1:** 15% RAP by weight of total mixture

**Tier 2:** between 16% and 25% RAP by weight of total mixture

**Tier 3:** greater than 25% RAP by weight of total mixture

In general, it is anticipated that mix design requirements for Superpave mixtures containing RAP should remain unchanged. As with virgin mixtures, the requirements for aggregate properties, gradation and volumetric properties should be met by the mix incorporating RAP. In simple terms, the aggregate portion of the RAP is handled as aggregate and the asphalt binder contained within the RAP is considered as asphalt binder contributing to the blended mixture. The grade of the asphalt binder selected is adjusted according to the proportion of RAP in the mixture. Table 5 outlines the tests required on the RAP and the associated selection of asphalt binder grade:

Numerous other recommendations were presented and designers should refer to the Superpave Mixtures ETG Guidelines for incorporation of RAP with Superpave. Furthermore, local experience with RAP should be incorporated when designing asphalt pavements with RAP.

**WHAT THE CHANGES MEAN…**

**… to Mix Designers,**

According to Paul Mack, Superpave Lead State team leader, the changes to the Superpave standards will provide numerous benefits to mix designers, contractors and the general public. For mix designers, the change from performing mix design at $N_{\text{max}}$ to

---

Table 5: Asphalt Binder Selection for Inclusion of RAP in Superpave

---
\(N_{\text{design}}\) will not only eliminate computational errors during backcalculation, but will also prolong the life of gyratory compactors by reducing the number of gyrations imposed on specimens. This in turn will reduce equipment costs. Furthermore, the new conditioning procedures will also reduce the time and number of ovens required during volumetric mix design, further reducing equipment costs and increasing productivity.

Ron Sines at the New York State Department of Transportation added that the addition of roadway descriptions to the \(N_{\text{design}}\) table will benefit municipalities that do not collect traffic data on a regular basis.

**... to Paving Contractors,**

The changes to density specifications at \(N_{\text{initial}}\) encourages Superpave mixtures that go over the restricted zone for low volume roads including local streets and even parking lots. Paving contractors will appreciate these finer mixtures, as they are easier to properly construct and compact.

**... to the Overall System,**

Overall, the changes in the Superpave standards integrate, consolidate and clarify the mix design procedure. By making the Superpave system easier to use, better performing asphalt pavements may be designed and constructed, providing long lasting and cost effective roads. Paul Mack summed up the changes by commenting that this new round of improvements “sets the stage for Superpave to be the HMA system” for all applications.

**THE FUTURE**

As mentioned, the results of NCHRP 9-12 should provide more comprehensive guidance for the inclusion of RAP in Superpave mixes. In addition, the incorporation of Stone Mastic (Matrix) Asphalt (SMA) mixture design into Superpave may be a future endeavour.

**REFERENCES**


Subcommittee on Materials Concurrent Letter Ballot. (Not available for circulation).

Mack, Paul J. “Improved Superpave Standards.” Published on the Lead States Website at [http://leadstates.tamu.edu](http://leadstates.tamu.edu).
APPENDIX C

RECOMMENDED GRADINGS FOR DIFFERENT MIX TYPES

Note: The Grading Limits and Nominal Mix Proportions given in this Appendix were obtained from the Standard Specifications for Road and Bridge Works for State Road Authorities, published by the Committee of Land Transport Officials (COLTO): 1998 Edition.
### Table C1. Grading Limits for combined Aggregate and Mix Proportions for Asphalt Bases

<table>
<thead>
<tr>
<th>SIEVE SIZE (mm)</th>
<th>SEMI-GAP-GRADED</th>
<th>CONTINUOUSLY GRADED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>37.5 mm MAX</td>
<td>26.5 mm MAX</td>
</tr>
<tr>
<td>53,000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>37,500</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>26,500</td>
<td>85 - 100</td>
<td>100</td>
</tr>
<tr>
<td>19,000</td>
<td>75 - 95</td>
<td>92 - 100</td>
</tr>
<tr>
<td>13,200</td>
<td>82 - 93</td>
<td>59 - 75</td>
</tr>
<tr>
<td>9,500</td>
<td>60 - 80</td>
<td>72 - 87</td>
</tr>
<tr>
<td>6,700</td>
<td>60 - 75</td>
<td></td>
</tr>
<tr>
<td>4,750</td>
<td>45 - 60</td>
<td>50 - 64</td>
</tr>
<tr>
<td>2,360</td>
<td>40 - 52</td>
<td>40 - 52</td>
</tr>
<tr>
<td>1,180</td>
<td>36 - 47</td>
<td>36 - 47</td>
</tr>
<tr>
<td>0,600</td>
<td>32 - 42</td>
<td>32 - 42</td>
</tr>
<tr>
<td>0,300</td>
<td>22 - 35</td>
<td>22 - 35</td>
</tr>
<tr>
<td>0,150</td>
<td>10 - 20</td>
<td>10 - 20</td>
</tr>
<tr>
<td>0,075</td>
<td>4 - 10</td>
<td>4 - 10</td>
</tr>
</tbody>
</table>

### NOMINAL MIX PROPORTIONS BY MASS WHEN BITUMEN IS USED

<table>
<thead>
<tr>
<th></th>
<th>Aggregate</th>
<th>93.5%</th>
<th>95.0%</th>
<th>94.5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bitumen (grade according to project specifications)</td>
<td>5.5%</td>
<td>4.0%</td>
<td>4.5%</td>
<td></td>
</tr>
<tr>
<td>Active filler</td>
<td>1.0%</td>
<td>1.0%</td>
<td>1.0%</td>
<td></td>
</tr>
<tr>
<td>SEIVE SIZE (mm)</td>
<td>NOMINAL MIX, MORTAR</td>
<td>Aggregate</td>
<td>BITUMEN GRADE ACCORDING TO PROJECT SPECIFICATIONS</td>
<td>ACTIVE FILLER</td>
</tr>
<tr>
<td>----------------</td>
<td>----------------------</td>
<td>-----------</td>
<td>-----------------------------------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>1180</td>
<td>0.80%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>600</td>
<td>0.60%</td>
<td>93.0%</td>
<td>8.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>300</td>
<td>0.30%</td>
<td>90.5%</td>
<td>8.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>150</td>
<td>0.15%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>110</td>
<td>0.11%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>75</td>
<td>0.75%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>45</td>
<td>0.45%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>25</td>
<td>0.25%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>12</td>
<td>0.12%</td>
<td>93.5%</td>
<td>5.5%</td>
<td>1.0%</td>
</tr>
<tr>
<td>5.12</td>
<td>0.05%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>5.12</td>
<td>0.05%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>4.12</td>
<td>0.04%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>4.12</td>
<td>0.04%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>4.12</td>
<td>0.04%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>4.12</td>
<td>0.04%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>4.12</td>
<td>0.04%</td>
<td>92.0%</td>
<td>7.0%</td>
<td>1.0%</td>
</tr>
</tbody>
</table>
Table C3. Grading Limits for Combined Aggregate and Mix Proportions for Non Homogeneous Modified Binders Continuously Graded Asphalt Surfacing

<table>
<thead>
<tr>
<th>Percentage passing sieve by mass</th>
<th>Sieve size (mm)</th>
<th>Percentage passing by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Continuously graded</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.2 mm max</td>
</tr>
<tr>
<td>19.0</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>13.2</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>9.5</td>
<td></td>
<td>80 - 100</td>
</tr>
<tr>
<td>4.75</td>
<td></td>
<td>50 - 70</td>
</tr>
<tr>
<td>2.36</td>
<td></td>
<td>32 - 50</td>
</tr>
<tr>
<td>1.18</td>
<td></td>
<td>13 - 25</td>
</tr>
<tr>
<td>0.60</td>
<td></td>
<td>8 - 18</td>
</tr>
<tr>
<td>0.30</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>0.15</td>
<td></td>
<td>4 - 8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nominal mix proportions by mass</th>
<th>Aggregate</th>
<th>Modified binder (bitumen rubber)</th>
<th>Active filler</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>91.0%</td>
<td>7.0%</td>
<td>2.0%</td>
</tr>
<tr>
<td></td>
<td>9.0%</td>
<td>7.0%</td>
<td>2.0%</td>
</tr>
</tbody>
</table>

Table C4. Grading Limits for Combined Aggregate and Mix Proportions for Conventional Non-Homogeneous Modified and Homogeneous Modified Bituminous Binders Open-Graded Asphalt Surfacing

<table>
<thead>
<tr>
<th>Percentage passing sieve by mass</th>
<th>Sieve size (mm)</th>
<th>Open graded asphalt mixes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>13.2 mm nominal</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Type 1</td>
</tr>
<tr>
<td>19.0</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>13.2</td>
<td>90 - 100</td>
<td>70 - 100</td>
</tr>
<tr>
<td>9.5</td>
<td>30 - 50</td>
<td>50 - 80</td>
</tr>
<tr>
<td>4.75</td>
<td>10 - 20</td>
<td>15 - 30</td>
</tr>
<tr>
<td>2.36</td>
<td>8 - 14</td>
<td>5 - 15</td>
</tr>
<tr>
<td>1.18</td>
<td>-</td>
<td>6 - 13</td>
</tr>
<tr>
<td>0.60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.30</td>
<td>-</td>
<td>3 - 8</td>
</tr>
<tr>
<td>0.15</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.075</td>
<td>2 - 6</td>
<td>2 - 5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Binder type</th>
<th>Penetration grade bitumen</th>
<th>Polymer-modified binder</th>
<th>Bitumen-rubber</th>
<th>Bitumen-rubber</th>
<th>As for 13.2 mm type 1 mixes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal mix proportions by mass</td>
<td>Aggregate</td>
<td>94.5%</td>
<td>94.0%</td>
<td>93.5%</td>
<td>93.5%</td>
</tr>
<tr>
<td></td>
<td>Binder content</td>
<td>4.5%</td>
<td>5.0%</td>
<td>5.5%</td>
<td>5.5%</td>
</tr>
<tr>
<td></td>
<td>Active filler</td>
<td>1.0%</td>
<td>1.0%</td>
<td>1.0%</td>
<td>1.0%</td>
</tr>
</tbody>
</table>
Table C5. Grading Limits for combined Aggregate and Mix Proportions for Conventional and Homogeneous Modified bituminous binders: Stone Mastic Asphalt Surfacing

<table>
<thead>
<tr>
<th>Percentage passing sieve by mass</th>
<th>Sieve size (mm)</th>
<th>13.2 mm max size</th>
<th>9.5 mm max size</th>
<th>6.7 mm max size</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.2</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5</td>
<td>67 - 90</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.7</td>
<td>41 - 65</td>
<td>50 - 80</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>4.75</td>
<td>30 - 50</td>
<td>30 - 55</td>
<td>80 - 100</td>
<td></td>
</tr>
<tr>
<td>2.36</td>
<td>21 - 32</td>
<td>22 - 32</td>
<td>34 - 44</td>
<td></td>
</tr>
<tr>
<td>1.18</td>
<td>17 - 27</td>
<td>17 - 27</td>
<td>24 - 34</td>
<td></td>
</tr>
<tr>
<td>0.600</td>
<td>14 - 24</td>
<td>14 - 23</td>
<td>18 - 30</td>
<td></td>
</tr>
<tr>
<td>0.300</td>
<td>11 - 23</td>
<td>11 - 22</td>
<td>13 - 25</td>
<td></td>
</tr>
<tr>
<td>0.150</td>
<td>9 - 17</td>
<td>9 - 19</td>
<td>9 - 19</td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>7 - 12</td>
<td>7 - 12</td>
<td>7 - 12</td>
<td></td>
</tr>
</tbody>
</table>

| Nominal mix proportions by mass  | Aggregate (%)   | 93.5             | 93.5            | 93.5            |
|                                  | Binder stabilizer (cellulose fibres) % | 0.3 - 0.5 | 0.3 - 0.5 | 0.3 - 0.5 |
|                                  | Binder content (%) | 6.5          | 6.5             | 6.5             |
APPENDIX D

A SIMPLE EXPERT SYSTEM FOR EVALUATION OF RUTTING POTENTIAL OF SAND-SKELETON MIXES
D.1 Introduction

The model presented here can be regarded as a simplified expert system for the evaluation of the rutting potential of densely graded stone-skeleton mixes. Specific aspects of the model are based on sound fundamental and statistical principles. However, the overall interpretation of all test results and the determination of a relative rut potential index rely strongly on expert interpretation.

D.2 Hypothesis Underlying the Expert System Model

In the following sections, the basic components of a model to evaluate and estimate rutting potential in a relative manner are described. This approach relies on the assumption that rutting resistance - at a specific temperature and load level - is determined by the following two factors (discussed in detail in Chapter 6):

i) Friction caused by aggregate interlock, and
ii) Cohesion caused by the binder and the mastic.

The friction factor is influenced by the spatial composition of the mix as well as by the aggregate characteristics. It is not dependent on temperature. The cohesive component is determined by the binder and is highly dependent on temperature. The complexity of the rutting phenomenon is, to a large extent, caused by the complex manner in which the friction and cohesive elements interact at different temperatures.

A single laboratory test such as the wheel-tracking test performed at different temperatures may be able to offer a combined estimate of all these factors. However, this would require that the test be implemented as a standard component of mix design - which is not considered feasible for most routine mix design projects. A wheel-tracking test can, however, be used to validate and calibrate the model during the later stages of development.

Thus the challenge is to find a way in which a relative evaluation of the two components listed above can be made without the need to perform advanced and potentially expensive wheel-tracking tests. There are two ways in which the cohesive and frictional components can be evaluated:

i) By relying solely on mechanistic tests such as ITT, ITS, dynamic creep and binder rheological tests, and

ii) By evaluating the results of mechanistic tests such as the ITT and dynamic shear rheometer in conjunction with important elements of spatial composition.

In view of the complexity of the rutting phenomenon and based on the findings of earlier literature surveys which suggested that - apart from wheel-tracking tests - there are few
routine mechanistic tests which adequately describe the rutting potential of all mix types, the latter approach is proposed.

The model form would therefore be as follows:

\[
\text{Rut Potential Indicator} = F(\text{environment, relative cohesion, relative friction})
\]

Environment is evaluated on the basis of temperature and traffic. The evaluation of cohesion is based on binder and mastic properties as well as on mechanical tests that are related to tensile strength (such as the ITS test). Friction is evaluated primarily by the dynamic creep test (see Chapter 8) and by evaluating aggregate and spatial composition properties.

### D.3 General Description of Expert System Approach

The specific parameters that are used to evaluate the environment, cohesion and frictional elements are discussed in detail in the following sections. In this section, the discussion focuses on the generic manner in which these parameters are evaluated to obtain an overall indication of rut resistance.

*The basic assumption of the model is that the relationship between individual test parameters (e.g. viscosity, ITS) and rutting potential is nonlinear and that there is a zone beyond which rutting potential increases considerably.* Unfortunately accurate and validated relationships between test indicators and rutting performance in the field seldom exist, mainly because of the lack of data.

To overcome the problem of scarcity of data, an expert system approach can be adopted. In this approach, threshold values (i.e. values beyond which rutting potential is expected to increase significantly) are estimated on the basis of expert knowledge and of analysis of the statistical distribution of observed test results.

As can be expected, threshold values that are determined in this manner will not be precise and accurate for all mix types. However, this element of uncertainty is alleviated by the fact that more than one type of indicator is used to estimate each component of mix strength. In this manner, the expert system model remains fairly insensitive to small variations in a single test parameter and only responds if one or more parameters exhibit extreme variations from acceptable norms. A key element of the approach is that the test parameter should have a well-accepted and validated relationship with rut potential. This issue is discussed in detail in the following section.
D.4 Aspects Related to Cohesive Strength

Viscosity at 60°C

Viscosity measurements provide a fundamental measure of the resistance of the binder to shear flow. The apparent viscosity, as measured in the Brookfield RV Viscometer, measures the ratio of the shear stress to the shear rate. The test conditions applied in the standard specifications do not apply equally to modified binders. However, the observed viscosity does provide an indication of the relative behaviour of a particular binder under selected test conditions.

The relationship between binder viscosity and shear flow that can lead to rutting is apparent from the definition of viscosity. Good correlation between binder viscosity and rutting, as measured under a wheel-tracking device, can, therefore, be expected. However, to take account of the potential changes that may occur during mixing and construction, it is recommended that the viscosity after ageing in the rolling thin film oven test (RTFOT) be used as input in the rutting model.

Table D.1 contains a statistical summary of viscosity measurements taken on a number of binders used for HMA construction. These results apply to binders that have been aged in the Rolling Thin Film Oven Test (RTFOT).
Table D.1: Statistical summary of Viscosity at 60°C values measured after RTFOT

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Viscosity at 60°C (Pa.s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum of observed range</td>
<td>181</td>
</tr>
<tr>
<td>25th Percentile of observed range</td>
<td>241</td>
</tr>
<tr>
<td>Average of observed range</td>
<td>302</td>
</tr>
<tr>
<td>75th Percentile of observed range</td>
<td>337</td>
</tr>
<tr>
<td>Maximum of observed range</td>
<td>492</td>
</tr>
<tr>
<td>Number of observations</td>
<td>38</td>
</tr>
</tbody>
</table>

Ring and Ball Softening Point

The ring and ball softening point provides an indication of the temperature at which a phase change occurs in the binder. Studies have shown that there is a clear relationship between softening point and rutting measured under a wheel-tracking test. This relationship may, however, not always be valid in the case of modified binders.

Table D.2 contains a statistical summary of softening point measurements taken on a number of binders used for HMA construction. These results apply to binders that have been aged in the rolling thin film oven test.

Table D.2: Statistical summary of Softening Point values measured after RTFOT

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Softening Point (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum of observed range</td>
<td>48.0</td>
</tr>
<tr>
<td>25th Percentile of observed range</td>
<td>52.3</td>
</tr>
<tr>
<td>Average of observed range</td>
<td>53.4</td>
</tr>
<tr>
<td>75th Percentile of observed range</td>
<td>54.2</td>
</tr>
<tr>
<td>Maximum of observed range</td>
<td>60.7</td>
</tr>
<tr>
<td>Number of observations</td>
<td>38</td>
</tr>
</tbody>
</table>

Indirect Tensile Strength

The indirect tensile strength (ITS) test is an inexpensive means of estimating the tensile strength of an asphalt mix. The test does not require dynamic loading and can be performed on field cores. The test gives an indication of the tensile strength of the mastic-aggregate combination and the test results can therefore be expected to correlate with the cohesive strength of the mix. Studies have shown that there is a relationship between permanent strain rate and ITS. An analysis of ITS values and corresponding rutting behaviour of some field projects suggests that the potential for poor rut performance is increased when the ITS values are less than approximately 1000 MPa. For high-traffic roads, rut behaviour seems to be consistently poor when ITS values are less than 800 MPa. This may, however, not always be valid in the case of modified binders.
Table D.3 contains a statistical summary of ITS values measured on a number of asphalt mixes. Some test results apply to field cores while others apply to mixes prepared in the laboratory using rolling wheel compaction.

**Table D.3: Statistical summary of ITS values measured on several HMA mixes**

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>ITS (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum of observed range</td>
<td>604</td>
</tr>
<tr>
<td>25th Percentile of observed range</td>
<td>893</td>
</tr>
<tr>
<td>Average of observed range</td>
<td>1095</td>
</tr>
<tr>
<td>75th Percentile of observed range</td>
<td>1219</td>
</tr>
<tr>
<td>Maximum of observed range</td>
<td>1745</td>
</tr>
<tr>
<td>Number of observations</td>
<td>33</td>
</tr>
</tbody>
</table>

**Vehicle Speed and Load Rate**

Asphalt behaves as a viscoelastic solid at most service temperatures. This means that the flow behaviour is dependent on the rate of loading as well as on temperature. At high loading rates (i.e. high traffic speeds) the binder stiffens, with a corresponding stiffening of the asphalt. At lower rates of loading or in the case of static loads, the binder softens, with a resultant increase in permanent deformation.

A layered elastic analysis of the stress influence lines in an asphalt surfacing was performed to assess the influence zones and speeds at which the load rate becomes significantly affected by vehicle speed. Figures D.2 and D.3 show the influence lines at different depths in a 100 mm thick asphalt surfacing supported by a medium stiff unbound base. The load used in this analysis was a single 20 kN wheel with a contact stress of 900 kPa.

It can be seen from Figures D.2 and D.3 that the stress influence zone for this wheel and pavement configuration is generally limited to an influence zone of approximately 1.0 m (i.e. 0.5 m each side of the point of investigation). Figure D.4 shows the time needed for the stresses to rise and fall over this distance, plotted against different vehicle speeds.

It should be noted that actual stress rise and fall times will vary, depending on the extent of the influence zone as well as on the depth of evaluation. However, it is clear from Figure D.4 that the rise and fall time of the stresses increases significantly at vehicle speeds below 20 km/h. This suggests that a critical threshold of 20 km/h may be indicative of increased rut potential.
Figure D.2: Stress Influence Lines for an Evaluation Depth of 10 mm

Figure D.3: Stress Influence Lines for an Evaluation Depth of 80 mm
D.5 Aspects Related to Frictional Resistance

Air Void Content and Voids Filled with Binder

The air void content and the voids filled with binder (VFB) are both related to the frictional resistance of the mix. There is considerable evidence that mixes tend to become unstable at air void contents approaching 2 per cent. At these low void levels, the mix becomes saturated with binder, with the result that the mix begins to flow as more of the load is carried by the mastic. This saturation of the mix with binder forces the aggregate skeleton apart and effectively reduces the frictional resistance. Furthermore, at high temperatures the mastic tends to act as a lubricant rather than as a cohesive element, which results in a further decrease in frictional resistance.

The VFB is closely related to the percentage voids in the mix and provides a combined indication of the void content and the voids in the mineral aggregate. Field studies have shown evidence that the rut potential of mixes is closely related to VFB. Threshold values of 4 per cent (minimum) and 70 per cent (maximum) for air void content and VFB, respectively, have been suggested.

Dynamic Creep Modulus

The Dynamic Creep test is discussed in Chapter 8. This test is a fairly inexpensive way of evaluating the frictional component of rut resistance, but it can be a time-consuming test, depending on the availability of suitable test equipment. It should, however, be noted that the applicability of the dynamic creep test is limited to densely graded sand-skeleton mixes, preferably manufactured with unmodified binders.
Table D.4 provides some guidelines for the interpretation of dynamic creep results:

Table D.4: Guidelines for the Interpretation of Dynamic Creep results

<table>
<thead>
<tr>
<th>Category</th>
<th>Dynamic Creep Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very good</td>
<td>&gt; 30</td>
</tr>
<tr>
<td>Good</td>
<td>15 – 30</td>
</tr>
<tr>
<td>Medium</td>
<td>10 – 15</td>
</tr>
<tr>
<td>Poor</td>
<td>&gt; 10</td>
</tr>
</tbody>
</table>

D.6 Aspects Related to the Environment

Temperature
Temperature is a key determinant of the environment in which the asphalt mix has to operate. The relationship between temperature and rut potential, however, is complex and is highly dependent on mix type and stress level. It would appear that temperature plays a greater role than stress level in determination of the rut rate. However, the influence of temperature may also be highly dependent on the applied stress. At higher stress levels, the influence of temperature on rut rate can generally be expected to increase.

Chapter 2 contains a map that provides a relative indication of temperature intensity levels for South Africa (cf. Figure 2.1). For the purposes of performing a relative evaluation of rutting potential, the following zones are defined (colour zones refer to the temperature map in Chapter 2):

- Zone 1: More than 1000 hours per year with pavement temperatures above 50°C;
- Zone 2: 500 to 1000 hours per year with pavement temperatures above 50°C;
- Zone 3: 250 to 500 hours per year with pavement temperatures above 50°C;
- Zone 4: Less than 250 hours per year with pavement temperatures above 50°C.

The ALS test results are largely supported by wheel-tracking test results. Figure D.5 shows the results of three tests conducted on a medium continuously graded mix and using Transportek’s wheel-tracking apparatus (cf. Chapter 6). The results of these three tests clearly suggest that the influence of temperature is significantly greater than that of load or pressure. It should, however, be noted that this observation might not be valid for all mix types. Mixes in which aggregate interlock does not properly develop under loading may exhibit a greater sensitivity to load magnitude.

Heavy Vehicles per Day
Together with temperature, the number of heavy vehicles per day is the key determinant of the relative aggressiveness of the environment in which a mix has to operate. For a given rut rate, the number of heavy vehicles per day determines the rut depth after a certain time. Table D.5 provides a relative indication of the intensity of heavy traffic loading.
Table D.5: Indication of Relative Intensity of Heavy Traffic

<table>
<thead>
<tr>
<th>Measure of Traffic Intensity</th>
<th>TRAFFIC CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Heavy Vehicles/Lane/Day</td>
<td>Approx. Pavement Structural Design Capacity</td>
</tr>
<tr>
<td>Less than 80</td>
<td>Less than 1.0 million ESALs</td>
</tr>
<tr>
<td>80 to 200</td>
<td>1.0 to 3 million ESALs</td>
</tr>
<tr>
<td>200 to 700</td>
<td>3 to 10 million ESALs</td>
</tr>
<tr>
<td>Greater than 700</td>
<td>Greater than 10 million ESALs</td>
</tr>
</tbody>
</table>

Figure D.5: Relative Influence of Load and Temperature on Wheel-Tracking Test Results

Load Intensity

The classification of traffic volumes shown in Table D.5 is based on traffic growth rates of less than 10 per cent, a design period of 15 years and assumes that only 50 per cent of heavy vehicles are fully laden. In addition to the evaluation of the pavement structural design class, the distribution of heavy vehicle volumes for typical provincial roads was also used to derive the guidelines set out in Table D.5.

The traffic classification shown in Table D.5 is therefore based on average, or typical design conditions. This classification has to be adjusted if the load intensity is likely to be increased by abnormal factors such as: (i) a high overload potential; (ii) a large percentage of fully laden or abnormally heavy vehicles (e.g. on mine haul roads), and (iii)
a traffic growth rate in excess of 10 per cent. For the purposes of obtaining a relative indication of rut potential, the following traffic intensity factors are proposed:

- Less than 50% of heavy vehicles fully laden: Intensity factor = 1
- 50% of heavy vehicles fully laden: Intensity factor = 2
- > 70% heavy vehicles fully laden: Intensity factor = 3
- Large percentage of overloaded or special loading conditions (e.g. on mine haul roads): Intensity factor = 4

The use of these factors for evaluation of rutting potential is explained in the following section.

D.7 Synthesis of Test Indicators for Evaluation of Rutting Potential

In the preceding sections the relationship between rut potential and various simple test parameters, as well as indicators related to the operating environment were explained. Tables D.6 and D.7 provide a simple model for synthesizing the information contained in the preceding sections.

**Table D.6: Relative Evaluation of Mix Rutting Resistance**

<table>
<thead>
<tr>
<th>Test Parameter</th>
<th>Relative Rutting Resistance</th>
<th>Weight Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High (4)</td>
<td>Med. to High (3)</td>
</tr>
<tr>
<td>Viscosity at 60°C after RTFOT (Pa.s)</td>
<td>&gt;340</td>
<td>300 to 340</td>
</tr>
<tr>
<td>R&amp;B softening point (°C) after RTFOT</td>
<td>&gt;55</td>
<td>54 to 55</td>
</tr>
<tr>
<td>Indirect Tensile Strength (kPa)</td>
<td>&gt;1220</td>
<td>1220 to 1100</td>
</tr>
<tr>
<td>Voids-filled-with-binder (%)</td>
<td>&lt;65</td>
<td>65 to 70</td>
</tr>
<tr>
<td>Dynamic Creep Modulus (MPa)</td>
<td>&gt;30</td>
<td>15 to 30</td>
</tr>
</tbody>
</table>

**Table D.7: Relative Evaluation of Rutting Potential Based on Environmental Considerations**

<table>
<thead>
<tr>
<th>Environment Parameter</th>
<th>Relative Rutting Potential</th>
<th>Weight Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low (1)</td>
<td>Low to Med. (2)</td>
</tr>
<tr>
<td>Temperature zone</td>
<td>Zone 1</td>
<td>Zone 2</td>
</tr>
<tr>
<td>Heavy vehicles/lane/day</td>
<td>&lt;60</td>
<td>60 to 200</td>
</tr>
<tr>
<td>Average heavy vehicle speed (km/h)</td>
<td>&gt;80</td>
<td>50 to 80</td>
</tr>
<tr>
<td>Traffic Intensity Factor</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>
The relative rutting potential can be determined by using Tables D.6 and D.7 in the following manner:

i) Determine as many of the available test parameters and environmental parameters as possible;

ii) For each parameter, mark the column in which the test parameter value falls. Multiply the value at the top of the relevant column by the weight factor shown in the last column and note the value. For example, if the ITS value is 1150, the “score” for that test parameter would be 3 (number at the top of column 2 in Table D.6) multiplied by 0.2 (weight factor for ITS) = 0.6.

iii) Add together all the scores for the evaluation of mix rut resistance (i.e. add scores for all rows of Table D.6). This total should be a number between 1 and 4.

iv) Add together all the scores for the evaluation of rut potential based on environment (i.e. the scores for all rows of Table D.7). This total should be a number between -1 and -4.

v) Add the scores obtained in steps (iii) and (iv). This should provide an indicator with a value between –3 and 3. A value of –3 is indicative of a mix with a very high rut potential and a value of 3 is indicative of a mix with low rut potential. For most situations, a positive indicator value would be indicative of a mix with a low rut potential.

It should be noted that the weight factors shown in Tables D.6 and D.7 are suggested values only. More experienced designers may wish to adjust these values to suit specific conditions. The weight factors should also be adjusted when one or more of the required test parameters are not available. Such adjustment of the weight factors can be made provided the weight factor values for Tables D.6 and D.7 add up to 1.0 and –1.0 respectively.

In the case of modified binders (e.g. bitumen blended with styrene-butadiene-styrene (SBS) or crumb rubber), the normal cohesion indicators do not always provide an adequate indication of rut potential. For mixes manufactured with such binders, it is recommended that the evaluation be augmented by means of wheel-tracking tests or axial load slab tests. In effect, this means that the relative evaluation scheme proposed here cannot be used on its own for the evaluation of mixes made with modified binders and that further testing should be considered for these mix types. This is not unreasonable, since modified binders are typically used on high-level design projects where more expensive performance tests such as wheel-tracking tests are justified.
APPENDIX E

MODEL FOR ABSOLUTE PREDICTION OF RUTTING UNDER TRAFFIC$^{23}$
E.1 Introduction

The model described in this Appendix is intended as a high-level analysis tool, which allows the rutting potential of a mix to be evaluated under very specific circumstances. Instead of a relative comparison of mixes at a single, fixed temperature and load level (as for example in the wheel-tracking test), the model described here can be used to evaluate the rut potential of a mix under specific temperature and loading conditions.

This approach allows mix designs to be optimized for specific site conditions, and also makes a prediction of the probable rut depth after different periods of trafficking. One disadvantage of the approach is that it requires more sophisticated tests. It also requires testing at different temperatures and load levels so that the performance of the mix under different conditions can be properly evaluated.

Details of the computer model and simulation process are described in the following sections.

E.2 Overview of the Modelling Approach

Figure E.1 shows an overview of the different inputs and tasks that are built into the prediction model. Each of the inputs A to E, as well as Tasks 1 to 4, are described in detail in the following sections. The prediction model attempts to simulate as closely as possible the conditions that are most likely to occur at different stages of the design period. The model thus requires reasonable estimates of the following:

- Pavement structural information (layer thickness and layer stiffness);
- Axle load distribution (i.e. the percentage of axles in different axle mass classes, together with their associated tyre contact stresses);
- Distribution of traffic during the day (i.e. percentage of total daily loading taking place in different periods of the day);
- Project location (identified by magisterial district and weather station name);
- For the design mix: the rut increment associated with different stresses, temperatures and number of loading repetitions (this information is obtained from the axial load slab test, described in detail in Chapter 8), and
- Temperature in the asphalt layer, at different times of the day as well as at different times of the year (i.e. seasonal and daily temperature variations need to be estimated).

Mechanistic design models typically use a standard axle load to calculate design stresses. However, the model described here uses the axle distribution (i.e. the number of axles in different axle mass classes, with their associated contact stresses). This information can typically be obtained from detailed traffic counts or, ideally, from Weigh-in-Motion (WIM) measurements.
For each axle class, calculate 100 typical stress values at the top of the asphalt layer in question (using layered elastic theory).

For each month and each daily period, estimate the temperature close to the top of the asphalt layer in question.

Determine typical rut increment for different temperature, stress level and number of load repetitions.

For each sub-period of each day of each month of the design period, do the following:

1. Obtain the temperature (for this daily sub-period and month) and also the total number of axles in each class (using information from Inputs B and D and Task 2).

2. Simulate the stresses applied by each axle in the daily sub-period by randomly selecting stresses from the array of typical stresses (using output from Task 1).

3. For each simulation of stress, increase a counter to keep track of the total number of simulated axles.

4. Use the selected stress, the daily sub-period temperature and the number of simulated axles to calculate the rut increment for each simulated axle. Add this to a running total to obtain an estimate of the accumulated rut.

Figure E.1: Structure of Stochastic Simulation Model

It should thus be clear that the absolute prediction model is somewhat complex and requires the designer to obtain detailed information on the expected traffic. While it is easy to scale down on the level of complexity, it should be recognized that this would necessarily have an impact on the accuracy of predictions.

For example, the daily temperature variation can be ignored and instead a single temperature (say, the maximum daily temperature) can be used to represent the dominant temperatures during each month. However, the maximum pavement temperature is likely to be recorded between 12h00 and 15h00. Only a small percentage of the total daily axle loading occurs during this period. Thus, the use of a single temperature to represent a typical day will result in an inaccurate estimate of rutting potential.

Similarly, if a standard axle load is used, the applied stress distribution is likely to be higher or lower than the actual stress distribution on top of the layer in question. Also, the use of a standard axle load does not take into account the high variation inherent in most traffic spectrums. Thus a certain level of complexity is unavoidable and any attempt at simplification is likely to result in reduced accuracy of rutting predictions.
E.3 Inputs Required for the Prediction Model

Pavement Structural Information (Input A in Figure E.1)

The pavement structural information required by the model consists of the stiffness, thickness and Poisson’s ratio of the different pavement layers. The PRORAS (Probabilistic Rut Analysis System) software estimates typical stiffnesses for each pavement layer based on the material type selected by the user. However, the user can change these stiffnesses to suit specific site conditions.

Layer stiffnesses are kept constant during the year and do not vary according to temperature and seasonal variation. This is a simplification that may have some impact on the calculated stresses at different times of the year. The stress parameter used in rut prediction is the vertical stress at the top of the asphalt layer. As may be expected, this parameter is affected by the stiffness of different layers, as shown in Table E.1.

The values shown in Table E.1 were calculated using layered elastic theory. The pavement structure consisted of a 40 mm asphalt surfacing with a 120 mm asphalt base, 150 mm granular subbase (300 MPa stiffness), 300 mm selected layer (120 MPa) and a semi-infinite subgrade (70 MPa). The Poisson’s ratio used for all layers was 0.4. The load used in the calculations was a single 40 kN load with a 800 kPa contact pressure.

<table>
<thead>
<tr>
<th>Asphalt Surfacing Stiffness (MPa)</th>
<th>Asphalt Base Stiffness (MPa)</th>
<th>Vertical Stress at top of Asphalt Base (kPa)</th>
<th>Change from base value*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>2000</td>
<td>860</td>
<td>17.6%</td>
</tr>
<tr>
<td>3000</td>
<td>2000</td>
<td>723</td>
<td>-1.1%</td>
</tr>
<tr>
<td><strong>2000</strong></td>
<td><strong>2000</strong></td>
<td>731</td>
<td>0.0%</td>
</tr>
<tr>
<td>2000</td>
<td>1000</td>
<td>735</td>
<td>0.6%</td>
</tr>
<tr>
<td>2000</td>
<td>3000</td>
<td>879</td>
<td>20.3%</td>
</tr>
</tbody>
</table>

* Base values used are shown in bold.

The error of up to 20 per cent shown in Table E.1 is considered to be within the variation expected as a result of variations in material density and stiffness over time and distance. At this stage, the improvement in model predictions that could be made by introducing temperature-sensitive material stiffnesses in the model is not considered to be cost-efficient, since such improvements would require stiffness tests to be performed at different temperatures. Future refinements of the model should, however, include a temperature sensitive stiffness model in the prediction of stresses.

Axle Mass Distribution and Daily Axles (Input B in Figure E.1)

The axle mass distribution defines the percentage of daily axles falling in different load classes. In the PRORAS software, the user can define up to 8 axle classes, with each
class having a different (user-defined) tyre contact pressure as well as a different user-defined ratio of super single to dual wheel load configuration. Table E.2 shows typical information required to define the axle mass distribution.

Table E.2: Typical Input Data Required to Define Axle Mass Distribution

<table>
<thead>
<tr>
<th>Axle Class*</th>
<th>Minimum Load (tons)</th>
<th>Maximum Load (tons)</th>
<th>Percentage of Daily Axles</th>
<th>Percentage of Super Singles</th>
<th>Tyre Contact Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>4</td>
<td>10</td>
<td>0</td>
<td>400</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>6</td>
<td>10</td>
<td>10</td>
<td>600</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>8</td>
<td>20</td>
<td>20</td>
<td>700</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>10</td>
<td>40</td>
<td>30</td>
<td>900</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>12</td>
<td>20</td>
<td>60</td>
<td>1000</td>
</tr>
</tbody>
</table>

* The PRORAS software allows a maximum of eight axle classes to be defined.

It can be seen from Table E.2 that the model allows for the analysis of the influence of overloading, which can be achieved by varying the percentage of axles with a mass in excess of 10 tons. The stochastic simulation will simulate the expected variation within each axle class by assuming that the loads within an axle class are normally distributed with a mean which is midway between the minimum and maximum axle loads for each class. If the user wishes to use a standard axle load, then only one axle class need be defined, with equal values for the minimum and maximum loads (i.e. no variation within the class is allowed).

The axle mass distribution should ideally be obtained from WIM surveys. For exploratory or sensitivity analysis, values can be approximated by using the traffic spectrum description obtained from detailed traffic counts.

In addition to the axle mass distribution, the model requires the total number of daily axles to be defined, as well as expected traffic growth percentage and a typical traffic wander distance. Traffic wander is defined as a standard deviation (in mm) from the centre of the wheel-path. This information is used in the stochastic calculation of typical stresses where a normal distribution is used to simulate traffic wander. Table E.3 shows typical wander distances associated with different lane widths.

Table E.3: Typical Wander Distances

<table>
<thead>
<tr>
<th>Average Lane Width (m)</th>
<th>Standard Deviation from Centre of Wheel Path (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.00</td>
<td>240</td>
</tr>
<tr>
<td>3.25</td>
<td>260</td>
</tr>
<tr>
<td>3.70</td>
<td>290</td>
</tr>
</tbody>
</table>

For a normal distribution, approximately 95 per cent of all observations lie within 2 standard deviations of the mean. This means that for a standard wander distance of 200 mm, 95 per cent of all wheels will pass within 400 mm of the centre of the wheel path (i.e. 95 per cent of all wheels will travel in a 800 mm wide zone).
Greater wander distances will cause the average stress observed at the centre of the wheel-path to be reduced. The use of a small wander distance will therefore result in a more conservative (i.e. higher) estimate of rut depth. It should be noted, however, that wander distances may decrease where traffic speeds are low (i.e. on climbing lanes). Wander distances have also been observed to decrease as the rut depth increases. It is therefore recommended that the wander distance be set to a maximum of 200 mm (standard deviation).

District and Weather Station Information (Input C in Figure E.1)
The expected temperature in the asphalt layer at different times of the day and year is a key input variable that has a significant impact on the predicted rut depth. For this reason a sophisticated prediction model is used to estimate daily and monthly temperature variations at a specified depth in the design asphalt layer. The procedure for the estimation of temperature is explained in detail in section E.5.

The PRORAS software is linked to a database containing typical monthly air temperatures for over 100 weather stations across South Africa. With time, this database will be expanded to include all available weather station data in southern Africa. In the PRORAS software, the user simply has to select the magisterial district in which the asphalt layer is to be built and then select the nearest weather station from the list of available weather stations in that area. The software then automatically extracts the monthly temperatures for that region from the database and calculates temperatures within the asphalt layer, using the procedure explained in Section E.5.

Percentage of Traffic in Each Daily Period (Input D in Figure E.1)
In the prediction model, each day is divided into 5 sub-periods. In the PRORAS software, these periods are defined as:

- Early morning: 04h00 to 08h00;
- Late morning: 08h00 to 12h00;
- Afternoon: 12h00 to 16h00;
- Early evening: 16h00 to 20h00;
- Nighttime: 20h00 to 04h00.

The model requires the percentage of the total daily heavy traffic falling in each of these daily sub-periods to be defined. This is a critical input segment, since the maximum temperature typically occurs in the afternoon (12h00 to 16h00), and the percentage of heavy traffic using the road during this period is bound to have an influence on the prediction of rutting.
Input of Axial Load Slab Data

The final input required before the simulation can be started is the definition of the rut increment as a function of temperature, vertical stress and number of load repetitions. This is done through the rutting function parameters obtained from the ALS test (described in Chapter 8). These parameters are derived through regression analysis and are provided as part of the ALS test result. The PRORAS software is linked to a database in which the results obtained with the axial load slab test can be stored. The software allows for the input of data into the database, as well as for the selection of results for different mix types for use in the simulation. The user can select up to three different materials that can be evaluated simultaneously and compared during the simulation.

E.4 Calculation of Typical Stresses Associated with Each Axle Class (Task 1 in Figure E.1)

Once the pavement structure, temperature, load and slab test information have been defined, the model is ready to perform the calculations needed for the simulation of rut development over the design period. These calculations comprise several tasks related to stress prediction, temperature prediction and actual simulation. The first task that has to be performed is the calculation of typical stress patterns induced by each axle class. This calculation is performed using a layered elastic model. The following steps are performed during the calculation:

i) The pavement structural information is obtained (defined in Input A).

ii) Using a standard normal distribution, a random load falling within the maximum and minimum axle mass limits is generated (defined in Input B).

iii) A standard normal distribution is used to generate a typical wander distance for the load in question.

iv) A random variable is used to determine whether the load is a super single or a dual wheel, using the proportions provided in Input B (see Table E.2)

v) The evaluation position is determined. This is calculated to be 5 mm from the top of the asphalt layer in question.

vi) The vertical stress at the evaluation position is calculated.

vii) The results are stored in an array.

Steps (i) to (vii) are repeated 100 times for each axle class. This results in an array with n columns and 100 rows, where n is the number of load classes. The 100 rows represent 100 typical stresses associated with each load class. This array is stored for later use during simulation.
E.5 Calculation of Monthly and Daily Temperature Variations (Task 2 in Figure E.1)

The procedure for the calculation of asphalt layer temperatures at different depths uses the mean maximum and minimum temperatures for each month to calculate the daily variations in temperature at a given depth within the asphalt layer.

The procedure consists of the following steps:

i) For the user-selected weather station, the mean maximum and minimum monthly air temperature, as well as the longitude and latitude, are extracted from the weather database.

ii) The day number to the middle of the month in question is determined, using a nominal 30 day month (e.g. for January, the day number will be 15, for February, it will be 45, etc.). Also determine the hour at the midpoint of each daily period (as defined in section E.3) is also determined.

iii) The zenith angle and the length of the day for the location (defined by weather station) and the day number are calculated.

iv) The maximum and minimum pavement surface temperatures are calculated by means of the appropriate equations.

v) The maximum and minimum asphalt temperatures close to the top of the design asphalt layer are calculated by means of the appropriate equations. (In the PRORAS software, the actual depth is taken as 5 mm for asphalt surfacings and, in the case of asphalt bases, as 2 mm into the base layer.)

vi) The temperature at the required depth (see step v) during the middle of each daily sub-period is determined, using a Sine-Exponential function to model the temperature variation during the day.

vii) This information is stored in an array.

Steps (i) to (vii) are repeated for each month of the year. The results are stored in an array for later use during simulation. This array has 12 rows (representing each month) and 5 columns (representing each daily sub-period). Thus, when Task 2 is completed, the temperatures at the midpoint of each daily sub-period are known for each month. These temperatures are calculated close to the top of the design layer and are used to represent the average temperature in the design asphalt layer.

E.6 Rut Increment Function (Task 3 in Figure E.1)

This task simply involves the extraction of the equation coefficients that define the rut increment as a function of temperature, vertical stress and number of load repetitions. The user selects the mixes for which the analysis has to be performed from a database of values. Three mixes can be evaluated simultaneously to facilitate a rapid comparison of performance. If the analysis has to be performed as part of a mix design, the model requires the axial slab test data for the design mix to be first entered in the database.
containing the test results. The user then selects the design mix as one of the mixes to be included in the simulation.

E.7 Simulation over Design Period (Task 4 in Figure E.1)

This task comprises the core of the simulation process. It requires the summation of rut increments over all axles that will travel over the design mix during the design period. Thus the simulation involves a summation over all axles of each sub-period of each day of each month of the design period. In the PRORAS software, the simulation is performed in such a manner that the axles loads are selected using the proportions specified in Input B.

An element of randomness is also inserted in the process by adding a random variation to the temperature calculated earlier in the calculation process (Step 2 in Figure E.1). This means that, although the monthly temperature variations will follow seasonal trends, the actual temperatures during each day will be slightly different.

E.8 Design Example

In the preceding section the input elements and steps needed to estimate the rut development over the design period were explained. In the following section, a typical design example is discussed, including some variations in the analysis to illustrate the influence that different input variables have on the predicted rut depth.

For this analysis, the following design parameters were assumed:
- Axles per day = 1000, with a distribution as shown in Table E.4;
- Traffic growth rate is 4 per cent per year, and
- Typical wander distance is 200 mm.

Two axle class definitions were analyzed. The first, defined as “case 1” in Table E.4, had 10 per cent overloading while the second (“case 2” in Table E.4) had 30 per cent overloading.

The analysis was performed for an asphalt surfacing with a thickness of 80 mm on top of a asphalt base. The subbase was assumed to be an intact cemented layer supported by a lime stabilized subgrade.
The ALS test results used for this example are for a continuously graded base with a stiff binder (penetration grade not known). By comparison with other mixes, this material had a relatively low rut rate and was fairly insensitive to stress level at temperatures below 40°C. At temperatures of 50°C and higher, however, it was highly sensitive to higher stress levels, at which the rut increment increased significantly.

Figure E.2 shows the predicted rut development for different scenarios. This figure shows two sets of lines. The two upper lines represent the prediction made for the Messina area, where the maximum daily pavement surface temperature are between 52°C and 58°C during the summer months. The two lower lines represent the prediction made for the Piet Retief area, where the maximum daily temperature during the summer months are much lower, being between 44°C and 50°C.

![Figure E.2: Predicted Rut Development for different Climatic Regions and Overload Potential](image)

It can be seen from Figure E.2 that the predicted rut depth for the Messina area (the two upper lines) is significantly higher than that for the cooler Piet Retief area. The effect of...
temperature generally overshadows that of load or overload potential. It is also interesting to note that the effect of overloading is greater for the warmer Messina area than in the case of the simulation for the Piet Retief area.

This simulation example illustrates some of the capabilities of the ALS test and the associated probabilistic analysis. If the PRORAS software is used for this analysis, the effect of many other elements on rut depth development can be studied. For example, the wander distance can be varied, as can contract stresses, traffic growth rates, layer thicknesses etc.