# Design and Use of Asphalt in Road Pavements
# Sabita Manual 35

February 2023

sabita

excellence in bituminous products

# Design and Use of Asphalt in Road Pavements

# SABITA MANUAL 35

Published by Sabita
Post Net Suite 56
Private Bag X21
Howard Place 7450
South Africa

ISBN 978-1-874968-71-9

**Revised Sixth Edition: February 2023**

**DISCLAIMER**

While considerable effort has been made to ensure the accuracy and reliability of the information contained in this publication, no warranty, assurance or representation is made in respect of fitness for purpose. Furthermore, neither Sabita nor any of its members can accept liability for any loss, damage or injury whatsoever resulting from the use of this information.

The user of this manual is deemed to have the necessary knowledge, skill and judgment to design bituminous surfacings and to exercise the necessary care and diligence in the application of this knowledge, skill and judgment  The information in this publication cannot supplant the user of the information's knowledge, skill and judgment based on the specific site and material conditions and state of the art and the user's exercise of the necessary care and diligence in the application of this knowledge, skill and judgment. Hence neither Sabita nor any of its members can accept liability for any loss, damage or injury whatsoever from the application, or failure to apply such knowledge, skill and professional judgment with the necessary care and diligence by the user of the information in this publication.

The above exclusions apply irrespective of any references in any COTO document to any of Sabita's manuals.

**Copyright Notice:** This document and the contents of these pages are the property of the Southern African Bitumen Association (Sabita). The contents may be reproduced in whole or in part for the purpose of implementation of the systems described herein, provided that the source is acknowledged. All other rights are reserved. Any other use requires the prior written permission of Sabita.

# Sabita manuals and DVDs

| | |
|---|---|
| Manual 1 | Technical guidelines:  Construction of bitumen rubber seals |
| Manual 2 | Bituminous binders for road construction and maintenance |
| Manual 5 | Guidelines for the manufacture and construction of asphalt |
| Manual 7 | SurperSurf – Economic warrants for surfacing roads |
| Manual 8 | Guidelines for the safe and responsible handling of bituminous products |
| Manual 10 | Bituminous surfacing for low volume roads and temporary deviations |
| Manual 12 | Labour Absorptive methods in road construction using bituminous materials |
| Manual 13 | LAMBs – The design and use of large aggregate mixes for bases |
| Manual 17 | Porous asphalt mixes:  Design and use |
| Manual 18 | Appropriate standards for the use of sand asphalt |
| Manual 19 | Guidelines for the design, manufacture and construction of bitumen rubber asphalt wearing courses |
| Manual 20 | Sealing of active cracks in road pavements |
| Manual 22 | Hot mix paving in adverse weather |
| Manual 23 | Code of practice:  Loading bitumen at refineries |
| Manual 24 | User guide for the design of asphalt mixes |
| Manual 25 | Code of practice:  Transportation, off-loading and storage of bitumen and bituminous products |
| Manual 26 | Interim guidelines for primes and stone pre-coating fluids |
| Manual 27 | Guidelines for thin hot mix asphalt wearing courses on residential streets |
| Manual 28 | Best practice for the design and construction of slurry seals |
| Manual 29 | Guide to the safe use of solvents in a bituminous products laboratory |
| Manual 30 | A guide to the selection of bituminous binders for road construction |
| Manual 31 | Guidelines for calibrating a binder distributor to ensure satisfactory performance |
| Manual 32 | Best practice guideline and specification for warm mix asphalt |
| Manual 33 | Design procedure for high modulus asphalt (EME) |
| Manual 34 | (A) Guidelines to the transportation of bitumen and (B) Bitumen spill protocol (booklets) |
| Manual 35 | Design and use of Asphalt in Road Pavements |
| Manual 36 | Use of Reclaimed Asphalt in the Production of Asphalt |
| Manual 37 | Sampling Methods for road construction materials |
| Manual 38 | A Health and Safety Guide for material testing laboratories in the road construction industry |
| Manual 39 | Laboratory testing protocols for binders and asphalt |
| Manual 40 | Design and construction of surfacing seals (*Due for completion by year-end*) |
| **TECHNICAL GUIDELINES** | |
| TG 1 | The use of modified binders in road construction |
| TG 2 | Bitumen stabilised materials |
| TG 3 | Asphalt reinforcement for road condition |
| TG 4 | Water quality for use in civil engineering laboratories |
| **DVDs** | |
| DVD 100 | Test methods for bituminous products |
| DVD 200 | Training guide for the construction and repair of bituminous surfacing by hand |
| DVD 300 | Manufacture, paving and compaction of hot mix asphalt |
| DVD 410 | The safe handling of bitumen |
| DVD 420 | Treatment of bitumen burns |
| DVD 430 | Working safely with bitumen |
| DVD 440 | Firefighting in the bituminous products industry |
| DVD 450 | Safe loading and off-loading of bitumen |
| (*The following manuals have been withdrawn:  3, 4, 6, 9, 11, 14, 15, 16 and 21*) | |

# Contents

## List of Tables

# List of Figures

# Preface

The purpose of this Sabita manual is to establish a common base for the design of asphalt mixes in South Africa. The intention is to advance the move towards performance-related specifications for the design of asphalt pavement materials, which started with the publication in 2001 of the *Interim Guidelines for the Design of Hot-Mix Asphalt* (*IGDHMA*) in South Africa. This move is in line with international best practice and also enables the formulation of national specifications that will reasonably ensure that asphalt layers will perform as expected.

Significant developments in asphalt technology have taken place since the publication of the IGDHMA and therefore a need existed to update the South African design methods for asphalt mixes, particularly in the light of the following developments:

- The revision of the *South African Pavement Design Method (SAPDM)* which allows for direct linkages between asphalt mix design, structural design and field performance in terms of resilient response and damage evolution. Previously, the design of asphalt mixes and the mechanistic-empirical design of the pavement structure were generally treated separately;
- The increasing use of mix types that cannot be classified as conventional Hot-Mix Asphalt (HMA) and that require alternative design methods. Such mix types would include warm mix, cold mix, mixes with significant proportions of reclaimed asphalt, stone mastic asphalt and Enrobé à Module Élevé (EME) asphalt. This is the reason for the shift in focus in this manual from HMA to asphalt in general;
- International and local advances in asphalt technology;
- Increase in volume of heavy vehicles on South Africa's roads;
- The need to supply roadway infrastructure for bus rapid transit systems;
- A demand for higher performance mixes, often leading to more sensitive mix designs;
- A need to review the current national compliance criteria for asphalt layers in contract specifications.

Furthermore, the methods proposed in the IGDHMA had never been properly validated. A need existed for a consolidated design manual containing well-validated methods to replace the existing guidelines.

This manual is based largely on research commissioned by Sabita and carried out by the CSIR Built Environment and completed in 2014. This research project comprised an extensive state-of-the-art study, consultations with industry experts; followed by laboratory investigations. The intention was to increase the reliability of the mix designs in terms of performance prediction, whilst at the same time simplifying the design process by reducing the number of test methods involved.

The February 2020 revised edition included reference to the newly published SABS technical specification SATS 3208: 2019 for Performance Grade bituminous binders, redefinition of the filler binder ratio as a simple mass ratio, reference to the COTO classification of grading classes for aggregates for asphalt mixtures, determination of the bulk density for dense mixes using the automatic vacuum sealing method – instead of the saturated-surface-dry method, assessment of the voids criteria for workability at 45 gyrations instead of 25 gyrations and redefinition of fatigue life in accordance with AASHTO T 321.

The February 2021 revision entails the revision of nomenclature and formulae associated with calculation of volumetric parameters, to bring them into line with the nomenclature adopted for the revision of the relevant SANS test standards and Sabita protocols for asphalt testing, as well as sequencing the calculations to reflect the order of laboratory procedures.

The July 2021 revision is aligned to the content of the imminent implementation of the PG specification for bituminous binders, as set out in SATS 3208, along with the removal of performance ratings of modified binders in terms of the compositional nomenclature used in TG1. Additionally the section of fatigue testing has been revised to provide for the rigorous application of AASHTO T321, in terms of selected strain levels and the criteria for the determination of fatigue life.

This, the fourth edition of May 2022, incorporates two significant amendments. Firstly, the determination of the bulk density of dense asphalt specimens shall now be determined by the vacuum sealing method according to AASHTO T 331, for both design and judgement of compliance purposes. Secondly, the compliance criteria for rutting, as measured in the Hamburg Wheel Tracking Test according to AASHTO: T 324, have been revised.

In this, the sixth edition of February 2023, revisions were made to target VMA values, as well as the relationship between layer or lift thickness and NMPS. In the previous editions of this manual VMA target values were based on the determination of volumetric parameters in accordance with SANS 3001-AS10 in conjunction with Marshall compaction equipment. With the shift of design voids to a target value of 4%, use of the vacuum sealing procedure (AASHTO T331) as the standard for determining bulk density as well as the gyratory compaction procedure for specimen preparation (AASHTO T 312) for design level IB and higher, an adjustment of the minimum target values of VMA was required to compensate both for bias between the results obtained with the two standards for bulk density determination as well as the degree of compaction. The revised target values of VMA, based on the assessment of a laboratory study, are presented as guidelines – and not specification requirements. Revisions to the relationship between layer or lift thickness and NMPS reflect the situation where the former, determined at the pavement structural design phase, or the latter (for practical considerations) will guide the selection of aggregate size during the mix design phase. Recommended minimum production volumes / plant run-time during plant mix trials are now included.

# Acknowledgements

We acknowledge the contributions made to this edition by the following persons.

| | |
|---|---|
| Joseph Anochie-Boateng | CSIR / University of Pretoria |
| Christi Botha | N3TC |
| Robson Francis | National Asphalt |
| Kevin Govender | Naidu Consulting |
| Phil Hendricks | Sabita |
| Wim Hofsink | WHCES |
| Emile Horak | JG Afrika |
| Kim Jenkins | University of Stellenbosch |
| Douglas Judd | N3TC |
| Julius Komba | CSIR |
| Tafadzwa Mafuma | AECI Much Asphalt |
| Joanne Muller | AECI Much Asphalt |
| Piet Myburgh | Consultant (Convenor) |
| Krishna Naidoo | SANRAL |
| Wynand Nortje | Shisalanga Construction (Pty) Ltd |
| Saied Solomons | Sabita |
| Benoit Verhaeghe | CSIR |

# List of abbreviations

| Abbreviation | Definition |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AMPT | Asphalt Mixture Performance Tester |
| AD | Apparent Density |
| A-E | Predominantly elastomer modified binder for asphalt |
| A-P | Predominantly plastomer modified binder for asphalt |
| A-H | Predominantly hydrocarbon modified binder for asphalt |
| A-R | Rubber modified binder for asphalt |
| ASTM | American Society for Testing and Materials |
| BD | Bulk Density |
| COLTO | Committee of Local Transport Officials |
| COTO | Committee of Transport Officials |
| DSR | Dynamic Shear Rheometer |
| E80 | Equivalent 80 kN axle load |
| EME | Enrobé à Module Élevé |
| EN | European Standard / Europäische Norm |
| EVA | Ethylene Vinyl Acetate |
| FBR | Filler / binder ratio |
| IGDHMA | Interim Guidelines for the Design of Hot-Mix Asphalt |
| ITS | Indirect Tensile Strength |
| LTPP | Long-Term Pavement Performance |
| MEPDG | Mechanistic-Empirical Pavement Design Guide |
| MPD | Mean Profile Depth |
| MVD | Maximum voidless density |
| NMPS | Nominal Maximum Particle Size |
| PCS | Primary Control Sieve |
| QC / QA | Quality Control / Quality Assurance |
| RA | Reclaimed asphalt |
| RTFOT | Rolling Thin Film Oven Test |
| SANAS | South African National Accreditation System |
| SANRAL | South African National Road Agency SOC Ltd |
| SANS | South African National Standards |
| SAPDM | South African Pavement Design method |
| SBR | Styrene-Butadiene Rubber |
| SBS | Styrene Butadiene Styrene |
| SMA | Stone Mastic Asphalt |

| | |
|---|---|
| TMH | Technical Methods for Highways |
| TRH | Technical Recommendations for Highways |
| TSR | Tensile Strength Ratio |
| USA | United States of America |
| UTFC | Ultra-Thin Friction Courses |
| VFB | Voids filled with Bitumen |
| VIM | Voids in the Mix |
| VMA | Voids in the Mineral Aggregate |
| WMA | Warm Mix Asphalt |

# 1. Introduction

The South African asphalt mix design methodology was updated and released in 2001 in the form of the *Interim Guidelines for the Design of Hot-Mix Asphalt* (IGDHMA). In 2005, the Sabita Manual 24: *User Guide for the Design of Hot Mix Asphalt* was published to supplement and support the use of the interim guidelines. The interim guidelines, as the name implies, were intended as a preliminary product, to be updated as the proposed methodology was validated.

The aim of this manual is to present a comprehensive, up-to-date design methodology applicable to asphalt mixes including conventional hot-mix asphalt, and special mixes (e.g., mixes produced at lower temperatures known as warm mix asphalts, Enrobé à Module Élevé (EME) asphalts, stone mastic asphalt porous asphalt, mixes intended for patching and pothole repairs, i.e. cold asphalt, mixes for light traffic in residential areas, and mixes with reclaimed asphalt and / or waste materials (e.g. slags). A more detailed mix design process and procedures for these special mixes are provided in various Sabita manuals and guidelines on the design of stone mastic asphalt is presented in this manual. All mixes are grouped into sand skeleton or stone skeleton categories based on their aggregate packing characteristics and, hence, gradings. The procedures used in this manual are in-line with the current international best practice.

The information contained in this manual has been compiled from various sources. These include the documents mentioned above, knowledge and experience recorded by the local asphalt industry and other institutions; experimental work and research studies undertaken by the CSIR and universities and both local and international published literature.

In this introductory chapter, the aims and scope of the asphalt manual are presented.

## 1.1 Aims of asphalt mix design

The purpose of asphalt mix design is to find a cost-effective combination of binder and aggregate, that is workable in the field, with sufficient binder to ensure satisfactory durability, fatigue performance and suitable aggregate configuration providing structure and space between particles to accommodate the binder and prevent bleeding and permanent deformation. If the material is used as a wearing course, the aim is to provide a surfacing that is waterproof (with the exception of porous asphalt) and meets functional requirements such as friction, noise attenuation and comfort. The intent of this manual is to assist mix designers in achieving this aim.

## 1.2 Performance-related asphalt mix design

The design philosophy in this manual follows the international trend, which is to move from a more empirical-based mix design approach towards the implementation of performance-related approach to set specifications for asphalt mixes. Performance specifications are based on the concept that mix properties should be evaluated in terms of the loading and environmental conditions that the asphalt material will be subjected to in service. The material parameters determined during the mix design phase should have a direct relation to the performance of the material in the pavement structure.

Performance-related mix design methods have been implemented in the USA in the form of the Superior performing pavements (Superpave) methodology. This is a move away from conventional asphalt mix design methodology in which empirical laboratory tests were used, which were only indirectly related to field performance. In Australia and New Zealand, the Austroads performance-related design method is used. The European Union has recently released the EN 13108 and EN 12697 standard series, as a step towards fully performance-related asphalt mix design. The move towards performance related design methods in South Africa is therefore in line with international developments.

## 1.3 Simplification

Previously, a range of test methods was used in the design of asphalt mixes in South Africa, often related to a single performance characteristic. It is not always possible to make meaningful comparisons based on a set of results obtained from different test methods for a single design parameter. Furthermore, it is a challenge to maintain current and well validated specifications for the material parameters for such a wide range of tests. Also, some routinely used test parameters have, at best, limited correlation to actual field performance (e.g. Marshall stability and flow).

Performance-related design methods aim to specify a limited number of performance criteria to be met by a mix design. In fact, the Eurocode prohibits the specification of more than one test per performance property (e.g. rutting), as this would represent over specification. This approach is taken further in this manual, as only a single test is described per performance indicator. The aim is to simplify the design process and to facilitate direct comparison of the performance of different mix designs. A reduction in the number of test methods also reduces the need for capital investment in laboratories.

## 1.4 Design approach

The intention of this manual is to replace the asphalt mix design methods in IGDHMA and related documents. Four levels of designs are used in relation to traffic volume and risk profile. A volumetric design approach is used to select optimum binder content for design situations with low to medium traffic levels (Levels IA and IB). The binder content obtained at this level serves as the starting point to select the optimum mix for design situations with moderately high to very high traffic volume with high level risk of structural damage (Level II and Level III). At these levels, the optimum binder content is selected based on performance-related tests.

Ultimately the traditional penetration grade binder selection will be replaced by performance grade binder selection methodology in which the binder is selected based on the loading and environmental conditions which the binder will be subjected to in service. It is the intention in this document to prepare the designer for this transition.

Selection of the design aggregate grading, determination of mix volumetrics, and moisture damage evaluation of the mix are the same for all levels of design.

There is a move away from grading bands to control points for aggregate design. The control points provided in this manual do not impose a restriction on the grading as per the current South African COLTO specifications. They are meant to be guidelines to develop the aggregate grading, rather than strict specifications. This distinction provides the designer with additional flexibility in adjusting aggregate gradings to meet volumetric requirements of the mix. The Bailey method, which has been used with success in South Africa, can be used to optimize aggregate grading and mix design criteria. For this reason an overview of the method is presented in Appendix A and the judicious application of the method merits serious consideration.

## 1.5 Link to pavement design

One of the shortcomings of the asphalt design methods previously available to South African practice was the lack of a relationship of the outputs of laboratory tests performed during the mix design and the performance characteristics of the mix in terms of elastic response, permanent deformation (rutting) and fatigue in a pavement structure.

Response and damage modelling in terms of dynamic modulus, and the resistance to fatigue and permanent deformation distress is safeguarded in the design methods presented to ensure that mixes will perform adequately in a range of applications, in terms of e.g. traffic and climate. However, to assign structural life to specific asphalt pavements in terms of actual environmental conditions and the composition of the total pavement structure falls beyond the ambit of this mix design manual.

In line with the approach to employ DSR testing on binders to gauge the effects of binder ageing on pavement performance, it is foreseen that, ultimately, the DSR will be used in a performance grade binder selection process, which will replace the conventional penetration grade framework. To advance the introduction of a performance grade binder specification, it is likely that rheological properties of binders based on DSR testing will be routinely reported, prior to adoption in a revised specification.

## 1.6 Accreditation of standard mixes

Where appropriate, mixes can be certified when they have gone through a process of comprehensive performance related testing. The certification will be associated with specific plant setups, materials (aggregate, filler and binder), their properties and mix characteristics such as binder content, voids in mix (VIM), voids in mix aggregate (VMA) and voids filled with binder (VFB). It is proposed that such a certification process be valid for a period of two years or such time during which any one of the mix components have not changed substantially.

The contractor can choose either to purposely design a mix to comply with the specifications, or select an existing mix design for which the properties are known. It is expected that the introduction of the performance-related mix design method will see the increased use of standard mix designs by producers and a reduction in the number of project specific mix designs.

In the European market it has become possible to get European Conformity (CE) markings for bituminous mixes, indicating that a product is fit-for-purpose. The CE certificate shows the product's performance in various performance-related tests. In South Africa, a similar system could be considered using Agrèment South Africa or another vehicle. Agrèment South Africa already provides fit-for-purpose certification for cold-mix asphalt and ultra-thin bituminous surfacing systems. Agrèment typically uses the services of independent South African National Accreditation System (SANAS) accredited laboratories for the required testing.

## 1.7 Scope and structure of the manual

This manual is intended to cover the design of all asphalt product types currently used in South Africa comprehensively. This includes: hot mix asphalts, warm mix asphalts, and EME asphalts, special designs such as stone mastic asphalt, porous asphalt, cold asphalt, mixes for light traffic in residential areas, and mixes with reclaimed asphalt.

In Chapter 2 of this manual, the process of selecting an appropriate mix type for each design situation is presented.

The performance-related binder selection methodology is presented in Chapter 3. The approach allows the selection of binders based on the combination of the environmental (climatic) and loading conditions under which the binder will be subjected to in the field. The temperature of the binder is determined based on locally developed temperature prediction algorithms.

Chapter 4 introduces aggregate selection based on the demands determined by the design situation.

Chapter 5 provides step-by-step procedures for the design and preparation of the asphalt mix. Depending on traffic volume and the risk level of structural damage, three mix design levels are presented in this chapter. Detailed design processes are presented for each level of mix design.

In chapter 6 some aspects on the interface between mix design and performance modelling are discussed. While the properties derived for mix design purposes may not be suitable for direct input into some more advanced pavement design models, they should provide assurance that mixes designed in accordance with this manual would be suitable for the prevailing conditions.

Finally, in Chapter 7, quality control and quality assurance for the best practice in asphalt manufacture and construction are presented, based on local experience and information from national specifications and various Sabita manuals are presented. Tolerances with regards to grading, binder properties, and volumetric properties are given. It is expected that gyratory compactors will be more widely distributed than is currently the case. The approach to quality control is divided into two categories:

- For low to medium volume roads where designs are more likely to be contract based;
- For medium to very high volume roads where mixes are more likely to be certified and control is exerted over the certified material and mix properties such as grading, VIM and binder content.

# 2. Mix type selection

## 2.1 Asphalt mix types

In this manual, asphalt mixes are primarily classified into two categories based on aggregate packing i.e. sand-skeleton or stone-skeleton types. Determining the aggregate packing characteristics of the mix is a critical choice to be made for mix type selection.

### 2.1.1 Sand skeleton mixes

In sand-skeleton mixes, the loads on the layer are carried mainly by the finer aggregate fraction, with the coarser aggregate fraction providing bulk and replacing a proportion of the finer fraction. The coarse aggregate fraction, in general, has little meaningful inter-particle contact and is not in a "compacted state" as would be the case for stone skeleton mixtures. Examples of sand skeleton mixes are continuously graded mixes as well as gap- and semi-gap graded mixes.

### 2.1.2 Stone skeleton mixes

In stone skeleton mixes the loads on the layer are carried by an interlocking matrix of the coarser aggregate fraction. For this structure to be realised, the spaces between the coarser aggregate fractions should not be over-filled by finer aggregate fractions and binder which would give rise to a situation where the coarser aggregates are being pushed apart. Avoiding such dilation of the coarse aggregate skeleton is critically important and will assure the functioning of the coarse aggregate fraction as the load bearing element. Examples of stone skeleton mixes include stone mastic asphalt, ultra-thin friction courses, and open graded (porous) mixes.

## 2.2 Factors impacting on selection of asphalt type

### 2.2.1 Traffic considerations

The following traffic aspects are considered in mix selection and design:

*Heavy vehicles*

For the purposes of mix design, traffic intensity / classes are evaluated using

Table 1: Traffic classification

| Design traffic [E80] [a] | Description |
|---|---|
| < 0.3 million | Low / Light |
| 0.3 to 3 million | Medium |
| >3 to 30 million | Heavy |
| > 30 to 100 million | Very heavy |
| > 100 million | Extreme |

*i. Axle loads*

Axle loads are limited to certain maximum values by law. The value of 80 kN is currently used as a standard in design calculations.

[a] E80 is an equivalent 80 kN axle load based on an exponential equivalency of 4,2. The standard axle load is an 80 kN single axle load with a dual wheel configuration

*ii. 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 % 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 which initiates more viscous behaviour of the binder, and increases the tendency for permanent deformation e.g. rutting in the wheel tracks. Mixes designed for climbing lanes, intersections or any other condition where heavy vehicle speeds are predominantly less than approximately 30 km per hour require special consideration.

*iii. Tyres*

Tyre construction, inflation pressures and tyre loading play a significant role in rutting and fatigue cracking in the asphalt material. Pertinent features are:

- Changes in tyre construction from cross-ply to radial ply have reduced fuel consumption by up to 30% by reducing the contact area, and, hence increasing contact pressure;
- By using fewer tyres and carrying heavier cargo, modern trucks are exerting much higher contact stresses on the road surface than their predecessors. If the tyre is under-inflated for the rated tyre loading, the tyre walls will exert significantly higher contact stress on the surface of the pavement relative to the centre of the tyre contact patch;
- On the other hand, higher tyre inflation pressures generally place greater contact stress on the asphalt layers (albeit to a lesser extent compared to the under-inflated case above) and therefore demand more stable asphalt mixes for these conditions.

#### *Light vehicles*

The volume and speed of light traffic need to be taken into account when functional properties such as friction, noise reduction and riding quality are being considered. High macro texture (or high mean profile depth - MPD) is required for mixes placed on roads where the speed of light traffic exceeds 60 km/h. Mixes placed in urban areas, where the volumes of light traffic are high, may need to have improved noise reduction properties.

Also, as densification of the layer under the action of light traffic is unlikely to be significant, initial impermeability (resistance of the asphalt layer to the passage of air and water into or through the mix) is an important consideration in the design and construction of such layers.

### 2.2.2 Braking and traction

At intersections or steep upgrades, braking and traction forces can be significant, leading to increased horizontal shear stresses and the potential for distortion or tearing of the layer. Some mixes may not be appropriate at intersections.

### 2.2.3 Fuel spillage

Spillage of fuel, particularly diesel, can cause softening of the asphalt, leading to distress which may not be representative of the mix behaviour and which cannot be predicted at the design stage. Where excess fuel spillage is expected it may be necessary to protect the asphalt layer or use a binder type, which is fuel resistant e.g. an EVA modified type.

### 2.2.4 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 e.g. on dedicated bus routes. In the latter situation, the degree of channelization is increased and consequently rutting resistance of the mix should be commensurate with the increased concentration of loading.

### 2.2.5 Layer or lift thickness and maximum particle size

The maximum aggregate particle size is a fundamental property of aggregate grading and asphalt mix type selection, and should be selected with due consideration of the intended asphalt layer thickness[b], and layer applications.

The selected maximum particle size for the asphalt mix should be determined by:

- Location of asphalt course in pavement;
- Proposed compacted thickness of layer, and
- Functional requirements of the asphalt layer.

Except for UTFC's and porous asphalt, it is generally accepted that the nominal maximum particle size (NMPS) should be less than $^{l}/_{3.5}$, where $l$ is the layer or lift thickness to ensure compactability and to counter segregation during paving. As an example, for a 45 mm asphalt layer, the NMPS should not exceed 10 mm or for a 30 mm layer the NMPS should not exceed 7.1 mm.

While asphalt layer thickness of a pavement is determined during the pavement structural design process, the construction of the layer may, for practical reasons, be carried out in more than one lift, with due consideration to:

- the construction equipment employed
- superior heat retention of thicker lifts to extend the compaction window in prevailing weather conditions
- the need for adequate cooling of the lift prior to:
  - the construction of subsequent lift; or
  - opening to traffic

Guidance on the selection of aggregate NMPS in relation to the lift thickness for layer construction is given in Table 2.

[b] In cases where layers are constructed in lifts, the criteria of the relationship of layer thickness and NMPS apply equally to lift thickness.

Table 2: Maximum NMPS values for selected layer or lift thickness

| Layer or lift thickness (mm) | Max value of NMPS (mm) |
|---|---|
| **20** | 5 |
| **25** | 7.1 |
| **30** | 7.1 |
| **35** | 10 |
| **40** | 10 |
| **50** | 14 |
| **60** | 14 |
| **80** | 20 |

Note that asphalt layers with thickness in excess of 80 mm are often constructed in two lifts.

### 2.2.6 Climatic considerations

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

*Maximum temperature*

Temperature is a key determinant for rutting potential. Maximum temperature influences the selection of mix type, aggregate type, and binder type.

*Intermediate and minimum temperatures*

These temperatures are determinants for fatigue and temperature fracture potential. For binders, intermediate temperature influences fatigue characteristics, and fracture potential is influenced by low temperature.

*Temperature differentials*

Temperature differentials increase the need for a balanced mix. Situations where extreme temperature fluctuations occur during the year increase the demand for a balanced, optimised asphalt mix which offers good resistance to rutting at high temperatures, as well as increased resistance to fatigue and temperature fracture at lower temperatures. Consideration should also be given to the selection of the binder type to guard against thermal fracture.

*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, and binder type.

### 2.2.7 Other considerations

*Functional requirements*

Special functional requirements may include:

- Mixes placed in urban areas, where light traffic volumes are high, may need to have improved noise reduction properties;
- Skid resistance requirements at relatively low speeds, and mean profile depth requirements at relatively high speeds, particularly, for high rainfall areas.

Skid resistance is primarily influenced by micro-texture and macro-texture of the aggregates in the road surface. The texture of the road surface influences friction developed between the tyre and asphalt surface to prevent skidding.

Table 3 defines classes of texture and their characteristics.

**Table 3: Classes of surface texture**

| Texture class | Amplitude of surface irregularity | Wavelength |
|---|---|---|
| **Micro-texture** | < 0.2 mm | < 0.5 mm |
| **Macro-texture** | 0.1 to 20 mm | 0.5 to 50 mm |
| **Mega-texture** | 0.1 to 50 mm | 50 to 500 mm |

The relationship of key vehicle operating and safety factors are illustrated in Figure 1.

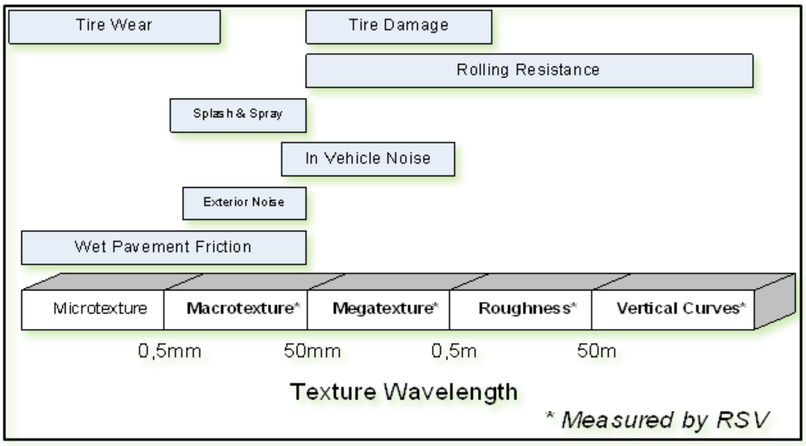


**Figure 1: Functional requirements in relation to surface texture**

## *Geometric conditions*

- Situations where braking, acceleration, crawling and turning of heavy vehicles are likely to occur on a regular basis require increased resistance to rutting, shoving, skidding and ravelling.
- Some difficulty may be expected in achieving specified finish tolerances and compaction at intersections, steep grades, and highly flexible supports; hence maintaining a minimum layer thickness would require special attention.

## *Material availability and project specifications*

- The availability of aggregates, filler and bitumen of the required quality should be evaluated before project specifications are finalised. Such evaluation at an early stage may lead to innovative practice in the interest of cost-effectiveness or 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 materials in the mix;
- The designer should ensure that component materials available from particular sources are of adequate supply, and can meet the project and product specifications. Materials

should preferably be obtained from a fixed commercial source. The properties of a material product supplied should not vary significantly during the supply period. In addition, the quality of the products should be such that it will not be negatively affected by transportation to site;

- Situations in which the standard specifications are modified to suit the needs of the project require special attention to be paid to availability and properties of local materials. Designers should alert tenderers to non-standard project specifications that may have an impact on material availability, especially situations in which locally available materials do not meet the project specifications;
- The decision to procure a material from a particular source depends on factors such as location of the source in the project proximity, availability of the required materials (in quality and quantity) from the source, as well as the economic consequences to the project;
- In some cases, to promote equitable tendering, the client is well advised to indicate nominal proportions of component materials, e.g. bitumen, filler and aggregates based on preliminary mix designs.

The aggregate types available from commercial sources commonly used for asphalt production in South Africa are given in Table 4.

**Table 4: Location of aggregates used in asphalt**

| Aggregate type | Province | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Eastern Cape | Free State | Gauteng | Kwazulu-Natal | Limpopo | Mpuma-langa | Northern Cape | North West | Western Cape |
| Andesite | | | ✓ | | | | | ✓ | |
| Dolerite | ✓ | ✓ | ✓ | ✓ | | ✓ | ✓ | | |
| Granite | | | ✓ | ✓ | ✓ | ✓ | ✓ | | |
| Greywacke/ Hornfels | | | | | | | | | ✓ |
| Norite | | | | | ✓ | ✓ | | ✓ | |
| Quartzite | ✓ | | ✓ | ✓ | | | | ✓ | ✓ |
| Tillite | | | | ✓ | | | ✓ | | |

**Note 2.1:** Certain mixes function well only when high quality components are used. Marginal or variable aggregates should not be used in mixes that are highly dependent on aggregate uniformity and interlock, such as SMA and porous asphalt. If aggregates are unlikely to provide sufficient deformation resistance owing to their shape characteristics, quality and variability, a binder of suitable rheological properties should be selected to reduce the potential for distortion of the asphalt layer.

Bitumen for asphalt manufacturing is either available from some SA refineries and secondary producers or suitable binders may be imported to meet demand. At the time of publication of this edition, bituminous binders are classified in terms of the mandatory national specification SANS 4001-BT1 for conventional bitumen graded in terms of its penetration. Also, the nomenclature for modified binders as set out in TG1: *The Use of Modified Bituminous Binders in Road Construction,* (November 2020) is still in use. As the implementation of the provisions of SATS 3208 takes effect, so the use of these categories would diminish until PG classification of binders will be implemented for general use.

Table 5 gives an indication of a likely relationship of conventional binder penetration grades and the PG classification framework. It needs to be added that the table content is based on somewhat limited test data and the correspondence given is by no means rigorous or exact. The relationship would typically depend on the source of the crude oil refined, refinery processes, source of the binder, the modification agents used as well as the compatibility of the modifier agent and the specific base bitumen used. Hence the reader is cautioned that the table merely serves as a guide and further testing would be required to establish the PG grade of a specific conventional binder.

In instances where conventional binders do not meet specific PG requirements such binders would require modification with e.g. elastomers, plastomers, reactive terpolymers or rubber crumbs[c] to meet the compliance requirements of SATS 3208. The use of a specific modifier would be determined by the manufacturer of the binder and would be based on binder / modifier compatibility, client specifications and economics.

**Table 5: Types of conventional bitumen for asphalt available from SA refineries and secondary producers**

| Penetration grade | 58S -22 | 58H -22 | 58V -22 | 64S -16 | 64H -16 | 70 -10 S-E |
|---|---|---|---|---|---|---|
| 70/100 | ✓ | | | | | |
| 50/70 | ✓ | ✓ | | ✓ | | |
| 35/50 | ✓ | ✓ | ✓ | ✓ | ✓ | |
| 10/20[d] | | | | ✓ | ✓ | ✓ |

The availability of appropriate crude sources and local demand may result in some refineries not producing some of the grades from time to time. Also, periodically, when local demand exceeds supply capacity and, given the limited bitumen storage capacity at refineries, bitumen is imported – either in bulk by ship or bitutainers or in drums.

## 2.3 Mix design considerations and mix type selection

The determination of aggregate packing characteristics of the mix (a stone-skeleton or a sand-skeleton type mix), are critical choices to be made for mix type selection in the mix design process. In doing so, consideration should be given to the following:

- The selected mix type ultimately determines the grading of the specific blend of aggregates used and typical grading types for various applications;
- Friction and noise are opposing properties except when open-graded asphalt and purpose designed friction courses are used;
- Thin layer asphalts for low speed and light traffic applications, mainly in residential areas are normally sand-skeleton type mixes;
- For mixes on high traffic volume applications, where friction properties and resistance to permanent deformation under elevated temperatures are key considerations, the preferred option is stone-skeleton type mixes;
- Continuous gradings that ensure sand-skeletons are frequently selected for general cases;

[c] Although not classified as a polymer, bitumen modified with crumb rubber displays characteristics similar to those associated with elastomers.

[d] This binder grade should additionally comply with the following additional requirements:

1. $G^*/\sin\delta$ at $T_{max}$ and @ 10 rad/sec and ≥ 4
2. % Recovery as per the MSCR method , 10%

- Continuously graded asphalt can be manufactured with grading varying from very coarse to very fine, for a particular maximum aggregate size;
- To ensure adequate skid resistance of gap-graded and semi gap-graded asphalt wearing courses, pre-coated chippings are usually spread on the freshly paved, hot mat prior to rolling;
- The practice of rolled-in-chips on continuously graded asphalt is not recommended in view of the possible adverse effect on mix performance in terms of durability and permeability. It is suggested that this practice should only be adopted where the asphalt manufacturer undertakes to design the mix with due consideration of these effects.

The selection of binders for specific applications will be influenced by both the critical required performance characteristics and available aggregate.  The optimal selection of a binder – conventional, or one modified with e.g. an elastomer (including rubber), terpolymer or plastomer – to address specific needs, should be based on laboratory design testing and cost implications, bearing in mind that meeting design criteria is influenced by the properties of both the binder and aggregate type and composition.

# 3. Binder selection

Binder selection for an asphalt layer should be supported by the following general considerations:

- Traffic;
- Climate;
- The modes of damage expected for the asphalt layer e.g., rutting, fatigue and ravelling. The expected modes of damage will most likely be influenced by historical modes of damage or expected future levels of traffic, substrate quality, climate or binder characteristics;
- Pavement structure and condition of the existing pavement, where appropriate; and
- Availability of binder and aggregate types.

The goal is to select a binder that will, in conjunction with the aggregate configuration, contribute to the performance of the asphalt under the prevailing conditions in such a manner as to provide the best “value for money.”

## 3.1 PG binder classification system

At the time of preparation of this edition, South Africa is in the process of transition from an industrial grade type bitumen specification to a performance grade (PG) specification. Since the compliance criteria for the various environmental and traffic situations are in the process of being formulated, an indication of a performance grade specification framework and related testing, likely to be implemented, is given in this document. As matters progress, the information in this manual will be updated. For the time being, the current specifications for binders generally used in asphalt mixes as given in SANS 4001-BT1 for penetration grade bitumen and in the Sabita Technical Guideline: *The use of modified bituminous binder in road construction, TG1* (November 2020) will apply where so required by client bodies. Alternatively PG binder specifications as set out in SATS 3208 may be introduced.

Performance grade specifications for binders focus on the evaluation of binder properties in terms of traffic loading and environmental service conditions – mainly temperature. The temperature of the asphalt layer (as determined by the climate), in conjunction with the grade (initial stiffness) and age of the binder, plays a pivotal role in determining the stiffness or dynamic modulus of the asphalt layer.

### 3.1.1 Temperature

The South African map depicting the 7-day average maximum asphalt temperatures at 20 mm depth is presented in Figure 2.

The maximum pavement design temperatures zones adopted for South Africa are 58°C, 64°C and 70°C.

While the minimum temperature in SA rarely falls below -10°C, the minimum temperatures adopted for grading purposes are considerably lower, to align the specification to the US standard and to determine the temperatures at which other tests are carried out i.e.:

- intermediate temperatures for fatigue (durability) and
- low temperatures for thermal fracture.

The three low temperatures associated with 58°C, 64°C and 70°C are -22°C, -16°C and -10°C, respectively i.e. an 80°C difference in all cases.

The maximum asphalt temperature zones are major determinants in the definition of a PG classification system.

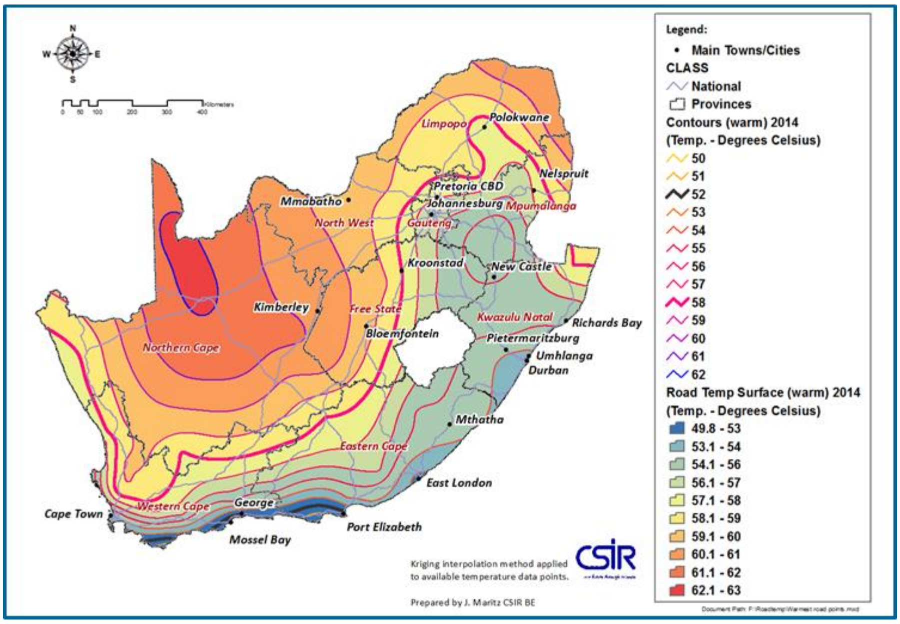


Figure 2: 7-day average maximum asphalt temperatures

In prescribing the temperatures at which tests are to be performed, the following benchmarks have been established:

- ***High temperature, $T_{max}$***: the applicable maximum pavement design temperature, e.g. 58°C, 64°C, 70°C
- ***Intermediate temperature, $T_{IT}$***: a temperature midway between $T_{max}$ and the minimum grading temperature $T_{min}$ plus 4°C, i.e. $[(T_{max} + T_{min})/2 + 4]°C$
- ***Low temperature: $T_{min}$***: 10°C above the minimum grading temperature, $T_{min}$, i.e. $[T_{min} + 10°]C$

### 3.1.2 Traffic

Traffic in the PG specification is classified both in terms of volume or severity and speed. This is done to take account of the fact that, for a given loading intensity, slow moving traffic would exert more severe loading conditions. It is proposed that six levels of traffic loading be adopted, in terms of E80's and ruling speed to provide a basis for binder selection.

As far as loading is concerned the design traffic categories are as follows:

< 0.3 million E80s
0.3 – 3 million E80s
> 3 – 10 million E80s
> 10 –30 million E80s
> 30 – 100 million E80s
> 100 million E8Os

Design speeds fall within the following categories:

< 20 km/h
20 – 80 km/h
> 80 km/h

Currently it is proposed that the combined effect of traffic loading and speed will be categorised in accordance with Table 6. In this table the following nomenclature is used:

- **S** –refers to standard conditions;
- **H –** refers to Heavy conditions;
- **V** –refers to Very heavy conditions, and
- **E –**refers to Extreme conditions

Table 6: Binder grade selection on the basis of traffic speed and volume

| Design traffic (million E80) | Traffic Speed (km/h) | | | Asphalt mix design level |
|---|---|---|---|---|
| | < 20 | 20 - 80 | >80 | |
| **< 0.3** | S | S | S | IA |
| **0.3 - 3** | H | S | S | IB |
| **> 3 - 10** | V | H | S | II |
| **> 10 - 30** | E | V | H | |
| **> 30 - 100** | E | E | V | III |
| **> 100** | E | E | E | |

The PG binder specifications for South Africa has been published by the South African Bureau of Standards (SABS) as a *technical specification* SATS 3208:2019. Salient compliance limits are given in

. For more details, the reader is referred to the SABS specification.

**Note 3.1**: The classification of traffic for *binder grade selection purposes* is distinct from that given for the selection of the mix design level, since traffic speed is taken into consideration in the former. The selection of mix design level and binder grade are therefore two separate processes.

Major advantages of the proposed PG grading include:

1. Improved prediction of asphalt mix performance is possible, thereby promoting more cost-effective design of mixes;
2. The effects of long-term ageing on performance of the binder, and hence the mix, can now be evaluated;
3. The specification is independent of the constitution of the binder in terms of type and quantity of modifier used and, hence, will promote cost effective use of costly modified binders; and
4. The specification is aligned to international practice.

Table 7: Binder classification and compliance limits

| Test property | Performance grade | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 58S -22 | 58H -22 | 58V -22 | 58E -22 | 64S -16 | 64H -16 | 64V -16 | 64E -16 | 70S -10 | 70H -10 | 70V -10 | 70E -10 |
| Maximum pavement design temperature, $T_{max}$ (˚C) | 58 | | | | 64 | | | | 70 | | | |
| Minimum grading temperature, $T_{min}$ (˚C) | -22 | | | | -16 | | | | -10 | | | |
| | Original binder | | | | | | | | | | | |
| G* and δ at $T_{IT}$, Pa, degrees | Compulsory report only during implementation phase | | | | | | | | | | | |
| G*/sinδ, at 10 rad/s, $T_{max}$, (kPa) | Compulsory report only during implementation phase | | | | | | | | | | | |
| Viscosity @ 165˚C (Pa.s) @ ≥30 $s^{-1}$, (Pa.s) | ≤ 0.9 | | | | | | | | | | | |
| Storage stability at 180°C (% diff, G*) at $T_{max}$ | ≤ 15 | | | | | | | | | | | |
| Flash Point (˚C) | ≥ 230 | | | | | | | | | | | |
| | After RTFO ageing | | | | | | | | | | | |
| G* and δ at $T_{IT}$, Pa, degrees | Compulsory report only during implementation phase | | | | | | | | | | | |
| Mass Change (m/m %) | ≤ 1.0 | | | | | | | | | | | |
| $J_{NR}$ @ $T_{max}$, ($kPa^{-1}$) | ≤ 4.5 | ≤ 2.0 | ≤ 1.0 | ≤ 0.5 | ≤ 4.5 | ≤ 2.0 | ≤ 1.0 | ≤ 0.5 | ≤ 4.5 | ≤ 2.0 | ≤ 1.0 | ≤ 0.5 |
| Ageing Ratio, $G^*_{RTFOT}/G^*_{Original}$ at 10 rad/s | ≤ 3.0 | | | | | | | | | | | |
| | After RTFO and PAV ageing | | | | | | | | | | | |
| G* and δ at $T_{IT}$, Pa, degrees | Compulsory report only during implementation phase | | | | | | | | | | | |
| Creep stiffness, S (60s) at $T_{min}$ + 10ºC , MPa, | ≤ 300 | | | | | | | | | | | |
| m (60s) at $T_{min}$ + 10ºC, minimum, MPa/s | ≥ 0.300 | | | | | | | | | | | |
| $\Delta T_c = T_cS - T_cm$ (ºC) | ≥ –5 | | | | | | | | | | | |
| Ageing Ratio, $G^*_{PAV}/G^*_{Original}$ | ≤ 6.0 | | | | | | | | | | | |

## 3.2 PG binder selection

Use of the PG binder classification system is self-explanatory, involving the following steps:

1. Locate the position of the asphalt layer on the map in Figure 2 indicating the 7-day average maximum asphalt temperatures at 20 mm depth.
    - If the asphalt layer is to be located wholly or partially within the > 58˚C Zone, a PG 64 binder is selected; or
    - If the asphalt layer is to be located wholly within the ≤ 58˚C Zone, a PG 58 is selected (a PG 64 will also conform to minimum requirements)
2. Determine the traffic level and average speed and choose the correct grade of binder as indicated in *3.1.2 Traffic*.

## 3.3 Binder selection for types of asphalt

The optimum binders applicable to various types of asphalt are dealt with in manuals dealing with the respective types and these are listed below. As mentioned above, bituminous binders are currently classified in terms of the mandatory national specification SANS 4001-BT1 and as set out in TG1: *The Use of Modified Bituminous Binders in Road Construction* is still in use. As the implementation of the provisions of SATS 3208 takes effect, so the use of these categories would diminish until PG classification of binders will be implemented for general use.

### 3.3.1 EME

"Enrobé à Module Élevé" or EME (high-stiffness asphalt for bases), using a very hard bitumen (ranging in penetration value from 10/20 to 15/25) is best for heavily trafficked applications where they provide excellent load spreading and are designed to have a 'perpetual' life.

The EME binder requirements are given in SANS: 4001-BT1. Currently work is underway to specify binders for EME in terms of the PG specification framework.

### 3.3.2 Sand asphalt

Refer to Sabita Manual 18 for details and the binder requirements for sand asphalt mixes.

### 3.3.3 Asphalt for lightly trafficked roads in residential areas

Refer to Sabita Manual 27 for the binder requirements for asphalt mixes in residential areas

### 3.3.4 Porous asphalt mixes

Refer to Sabita Manual 17 for the binder requirements for porous asphalt mixes.

### 3.3.5 Bitumen rubber asphalt

Refer to Sabita Manual 19 for the binder requirements for bitumen rubber asphalt.

### 3.3.6 Warm mix asphalt

Refer to Sabita Manual 32 for the binder requirements for warm mix asphalt.

**Note 3.4:** It is important that the final function of the binder is not negatively influenced by the WMA additives, and if the binders are to be evaluated, it must be done with the additives already present.

**Note 3.5:** There may be a need under certain circumstances to specify a harder grade of warm mix binder. This is due to the fact that warm mix binders will undergo less ageing and oxidative hardening during manufacture and laying and, as a result, some warm mixes have shown a reduced resistance to rutting.

## 3.4 Reclaimed asphalt binder

The effective binder grade after blending with the reclaimed asphalt binder and any rejuvenating agents should be specified for the contract. Practically, this may be determined beforehand by blending virgin binder, binder recovered from the recycled asphalt and rejuvenator in the theoretical proportions and evaluating the blended binder. Alternatively, the final binder grade may also be estimated using the so-called "mortar" test, described in AASHTO Designation: T XXX-12.

Experience has shown that the PG grading classification system may be more suitable for the testing of RA binders.

**Note 3.6:** Care should be taken to specify the effective binder grade according to the expected paving conditions and the amount of ageing of the binder expected to occur. For example, if the rejuvenating agent is also a warm mix additive, one may specify a harder effective binder grade to compensate for the reduced amount of aging the binder will undergo, as some warm mixes have been shown to have reduced resistance to rutting.

# 4. Aggregate Selection

## 4.1 Aggregate materials

Aggregate consists of hard material which is generally derived from the crushing of solid rock or boulders. As aggregates constitute approximately 95% of the mass and 85% of the volume of continuously (dense) graded asphalt mixes, the structural and functional performance of an asphalt mix in the pavement layer is largely influenced by the physical properties and characteristics of the aggregate blend.

## 4.2 Definitions

Aggregate materials for asphalt mix designs are mainly divided into three sizes (coarse aggregates, fine aggregates, and fillers), and are conventionally defined as follows:

- Coarse aggregates (crushed rock, crushed blast-furnace slag, etc.) – materials retained on the 5 mm sieve[e];
- Fine aggregates (crusher sand, clean natural sand, mine sand, selected river gravel or a mixture of these.) – materials passing the 5 mm sieve but are retained on the 0.075 mm sieve;
- Filler – materials passing the 0.075 mm sieve.

## 4.3 Aggregate sources

### 4.3.1 Natural aggregates

Natural aggregates are used in their natural form. They are mined from river, Aeolian or glacial deposits and are used without further processing to manufacture asphalt mixes. The two commonly used natural aggregates for asphalt mixes are gravel and sand. Aeolian deposits in particular comprise mostly rounded particles, which may promote workability on the one hand, but compromise the mixes resistance to permanent deformation on the other.

### 4.3.2 Processed aggregates

Processed aggregates have been quarried, crushed and screened in preparation for use. These aggregates are processed to achieve certain performance characteristics of the manufactured asphalt. It is desirable to have cubic and angular crushed aggregates for asphalt mix design. Particles that are flat, elongated, or both, can adversely affect the composition and performance of an asphalt mix.

#### *Manufactured aggregates*

Manufactured aggregates may be either by-products of an industrial process, such as industrial slag, calcined bauxite, or products specifically obtained and processed for use as aggregates (e.g. reclaimed asphalt, recycled concrete aggregate).

#### *Slag aggregates*

The two main types of slags available for use in asphalt mixes are steel and ferro-chrome.

Steel slag is a by-product of the steel making process. Utilising steel slag as an aggregate is a means to reduce the large waste stockpiles, as well as to preserve natural resources by not quarrying natural aggregates. The pH is between 8 and 11, and hence it has a strong affinity to bitumen which

[e] In SMA, which consists of a binary system of aggregate and mortar, the coarse aggregate is deemed to be that which is retained on the 2 mm sieve; the balance being the fine material, which together with the filler makes up the mortar.

aids in retaining the binder coating and preventing stripping. This benefits long-term durability, especially in high moisture regions.

Water absorption of ferro-chrome slag is relatively high due to blow holes in its structure. This may lead to slightly higher binder content due to some binder being lost in these blow holes. However, there are no micro fissures in the slag as in some natural aggregates with high absorption, so that selective absorption of the bitumen is not considered to be a problem

**Note 4.1:** Before using steel slag as an aggregate in asphalt, it is critically important that it is weathered prior to use in order to prevent expansion. The purpose is to hydrate the free calcium oxide, which, if not done, results in water causing hydration and breaking down of the aggregate. It is a recommendation that steel slag for road construction aggregate should be stockpiled for a minimum of three months and kept constantly wet by water spraying.

#### *Reclaimed asphalt (RA) aggregate*

RA consists of fragments of asphalt that have been removed from the road or sourced from stockpiles of discarded asphalt. Guidelines for sampling of aggregate materials (TMH5 C5) can be followed to sample RA from a stockpile. Segregation is generally a major concern when sampling from RA stockpiles, and care must be taken to avoid it. Processing of RA should be based on recommendations provided in TRH 21.

**Note 4.2:** When 20% or more RA is used in asphalt, testing of the RA aggregate and the aged binder is recommended.

### 4.3.4 Fillers

Fillers are essential for producing asphalt mixes which are dense, cohesive, durable and resistant to water penetration. Filler consists of:

- Inert fillers, such as natural dust or rock-flour; and
- Active fillers like hydrated lime or cement.

In an asphalt mix, the filler generally serves the following purposes:

i. Acts as an extender for binder to stiffen the mastic and the mix, thereby improving stability.
ii. Acts as a void-filling material which can be used to adjust gradings and volumetric properties.
iii. Some fillers e.g. lime are used to improve the bond between the binder and the aggregate.
iv. Specific fillers such as fly ash can be used to improve mix compactability.

Adequate amounts of filler ensure adequate cohesion, which is a major contributing factor to the provision of resistance to permanent deformation especially in sand-skeleton mixes. Too much filler stiffens the mix, and the mix will be difficult to compact, and too little will result in low cohesion, and the mix may fall apart.

Table 8 summarises filler types, characteristics and test methods to determine their properties.

Table 8: Filler types and characteristics

| Type of filler/origin | Characteristics | Test method / Criteria |
|---|---|---|
| Hydrated lime (active filler) | • Improves adhesion between binder and aggregate<br>• Improves mix durability by retarding oxidative hardening of the binder<br>• Low bulk density and high surface area<br>• Relatively high cost<br>• Monitor effect on stiffness to ensure compactability | • Grading (% passing 0.075 mm) (SANS 3001-AG1): minimum 70<br>• Bulk density in toluene (BS 812): 0.5 – 0.9 g/ml<br>• Voids in compacted filler (BS 812): 0.3 – 0.5% |
| Portland cement (active filler) | • Relatively high cost<br>• Monitor effect on stiffness to ensure compactability | |
| Baghouse fines | • Variable characteristics require control<br>• Some source types may affect mix durability<br>• Some types may render mixes sensitive to small variations in binder content | Methylene blue test (SANS 6243): maximum value 5 |
| Limestone dust | • Manufactured under controlled conditions and complies with set grading requirements<br>• More cost-effective than active filler<br>• Although it is viewed as an inert filler, the high pH value reduces moisture susceptibility | Methylene blue test (SANS 6243): maximum value 5 |
| Fly ash (non-active filler) | • Low bulk density<br>• Relatively high cost<br>• Variable characteristics require greater control | Same test methods as for active fillers (above) |

**Note 4.3:** The binder-with-filler component may stiffen dramatically beyond a certain filler-binder ratio (FBR), i.e. the ratio $\frac{mass\ of\ filler}{mass\ of\ binder}$. It is recommended that the filler-binder ratio of surfacing 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 maximum ratio of approximately 1.6). Also, in open-graded mixes such as UTFC, consideration can be given to higher permissible filler-binder ratios; up to 1.7%.

**Note 4.4:** When active fillers such as cement and hydrated lime are used care should be taken not to increase the viscosity of the hot mastic beyond values that will adversely affect workability during mixing and paving. Where hydrated lime is used the quantity should be limited to 1% by mass of the total aggregate.

**Note 4.5:** Small increases in the amount of filler in grading can literally absorb much of the binder resulting in a dry unstable mix, and small decreases, i.e., too little filler will result in too rich (or wet) mixes.

## 4.4 Aggregate shape

Aggregates suitable for use in asphalt should be cubical rather than flat, thin or elongated. These particles exhibit greater interlock and internal friction resulting in greater mechanical stability of compacted mix. Mixes containing rounded particles have better workability and require less compactive effort to obtain the required density. This ease of compaction is not necessarily an advantage because mixes that are easy to compact during construction may continue to densify under traffic and resulting in excessive rutting due to low voids. For this reason a maximum field density is proposed in Table 33, Table 34,

Table 36 and Table 37.

## 4.5 Aggregate grading

In determining the grading of an aggregate, a sample of the material is sieved through a nest of sieves and the percentage by mass of material passing each sieve is determined. The SANS 3001-AG1 procedures will be followed in this manual for particle size analysis of aggregates by sieving. Table 9 shows the comparative sieve sizes for aggregate grading in South Africa. Sieve sizes as per SANS 3001 are used in this document.

**Table 9: Changes in sieve sizes from TMH1 to SANS**

| TMH 1 sieve sizes [mm] | SANS 3001 sieve sizes [mm] |
|---|---|
| 37.5 | 37.5 |
| 26.5 | 28 |
| 19 | 20 |
| 13.2 | 14 |
| 9.5 | 10 |
| 6.7 | 7.1 |
| 4.75 | 5 |
| 2.36 | 2 |
| 1.18 | 1 |
| 0.6 | 0.6 |
| 0.3 | 0.3 |
| 0.15 | 0.15 |
| 0.075 | 0.075 |

### 4.5.1 Coarse aggregate grading classes

The coarse aggregate components of any asphalt mixture should comply with the grading limits tabled in Table A9.1.5-3 of COTO (given below in Table 10) for the relevant grading class and NMPS as listed. The grading class applicable to a specific mix type is as follows:

*Grading class 1:*

- Gap graded: stone skeleton mixes e.g. Stone mastic asphalt (SMA)
- Ultra-thin friction courses (UTFC)
- Porous asphalt

*Grading class 2:*

- Continuously graded: sand skeleton mixes
- Semi-gap graded sand skeleton mixes
- Gap graded sand skeleton mixes
- High modulus asphalt (e.g. EME)

**Table 10: Grading limits for nominal size coarse aggregates**

| | Nominal maximum particle size (NMPS) (mm) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 28,0 | | 20,0 | | 14,0 | | 10,0 | | 7,1 | | 5,0 | |
| Grading class | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 |
| Sieve size (mm) | Percentage passing sieve size by mass | | | | | | | | | | | |
| 37,5 | 100 | 100 | - | - | - | - | - | - | - | - | - | - |
| 28,0 | 85-100 | 85-100 | 100 | 100 | - | - | - | - | - | - | - | - |
| 20,0 | 0-20 | 0-35 | 85-100 | 85-100 | 100 | 100 | - | - | - | - | - | - |
| 14,0 | 0-5 | 0-5 | 0-20 | 0-35 | 85-100 | 85-100 | 100 | 100 | | | | |
| 10,0 | | | 0-5 | 0-5 | 0-20 | 0-35 | 85-100 | 85-100 | 100 | 100 | | |
| 7,0 | | | | | 0-5 | 0-5 | 0-20 | 0-35 | 85-100 | 85-100 | 100 | 100 |

| 5,0 | | 0-5 | 0-5 | 0-20 | 0-35 | 85-100 | 85-100 |
|---|---|---|---|---|---|---|---|

### 4.5.2 Fine aggregate grading

Grading limits for the fine aggregate components of an asphalt mixture should comply with Table A9.1.5.-5 of COTO (given below in Table 11). In some cases it may be expedient to permit the use of a natural fines component – not obtained from crushed parent rock – provided, of course, that the specified mix properties are met. It is good practice to limit the proportion of such natural fines to 7% by mass of the fine aggregate. In addition the liquid limit should not exceed 25 % and the PI not exceed 4. Natural fines should be introduced into the mixing plant by means of a separate plant cold-feed bin.

Table 11: Fine aggregate grading limits for relevant mix type

| Aggregate Class | Class 1 | Class 2 | Stockpile Tolerance |
|---|---|---|---|
| | Percentage passing by mass | | |
| Sieve Size (mm) | Stone skeletal mixes as defined | Sand skeletal mixes as defined | |
| 7 | 100 | 85 - 100 | 5% |
| 5 | 90 - 100 | 70 - 90 | 5% |
| 2 | 65 - 90 | 45 - 70 | 5% |
| 1 | 45 - 70 | 28 - 50 | 5% |
| 0.6 | 30 - 50 | 19 - 34 | 5% |
| 0.3 | 18 - 30 | 12 - 25 | 4% |
| 0.15 | 10 - 21 | 7 - 18 | 3% |
| 0.075 | 5 - 15 | 5 - 15 | 2% |

The Fineness Modulus (FM) of both the crushed and natural fine aggregates should not deviate by more than 0.2 from that determined on the fine aggregate incorporated in the approved design mix.

## 4.6 Mix grading requirements

### 4.6.1 Grading control points

To achieve suitable aggregate packing to ensure that relevant performance characteristics of a particular mix are met, aggregates of various sizes are mixed in certain proportions, Such proportions are defined by the particle shape, texture and size distribution as represented by a grading. This grading will then be used primarily as a quality assurance measure to ensure that the intended packing features are achieved and maintained for a particular aggregate type.

To guide designers, especially when preparing a first-off design with specific aggregates in a particular application, some guidelines are offered here. It is suggested that the grading of an aggregate blend should lie within certain key control points as follows:

- The nominal maximum particle size (NMPS) – designated as one sieve size larger than the largest sieve to retain a minimum of 15 percent of the aggregate particles –should be selected in accordance with Table 2.
- The 2 mm sieve, and the 0.075 mm sieve.

Table 12 provides grading control points for four nominal maximum particles sizes of aggregates typically used for production of ***sand skeleton*** **(continuously graded)** asphalt mixes in South Africa. The control points for 20 mm NMPS are plotted on a 0.45 power chart in Figure 3 for illustration purposes.

**Note 4.7:** The control points given in Table 12 ***should be used as guidelines only and are not relevant to mixes such as stone skeleton types (including SMA)*** in which cases it is suggested that specific methods of aggregate proportioning, such as the Bailey method, as set out in Appendix A, or the volumetric principles as set out in Appendix B – Principles of the Design of Stone Mastic Asphalt, need to be employed.

**Note 4.8:** The gradation of continuously graded asphalt should not be too close to the 0.45 power maximum density curve. If it is, then the VMA is likely to be too low leading to low binder content to attain minimum voids in the mix. Gradation should deviate from this maximum density curve, especially on the 2.00 mm sieve.

To optimise aggregate proportions, it is recommended that designers consider the use of the Bailey method[f], which has been used with success in heavy duty asphalt applications in South Africa. The designer should be mindful of the fact that some parameters of this method are based on aggregates encountered in the USA. However these ratios have been reviewed in the light of the SANS sieve sizes that came into effect in 2013; consequently the method provides valuable guidance to determining the optimal proportioning of asphalt mixes for a wide range of applications and will instil a clearer understanding of aggregate packing configurations that are not evident in particle size distributions.

An overview of the method is provided in Appendix A.

**Table 12: Aggregate grading control points**

| Sieve sizes [mm] | Percent passing nominal maximum particle size (NMPS) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | NMPS = 28 mm | | NMPS = 20 mm | | NMPS = 14mm | | NMPS = 10 mm | |
| | Min | Max | Min | Max | Min | Max | Min | Max |
| **37.5** | 100 | | | | | | | |
| **28** | 85 | 100 | 100 | | | | | |
| **20** | | 85 | 85 | 100 | 100 | | | |
| **14** | | | | 85 | 85 | 100 | 100 | |
| **10** | | | | | | 85 | 85 | 100 |
| **7.1** | | | | | | | | 85 |
| **5** | | | | | | | | |
| **2** | 19 | 45 | 23 | 49 | 28 | 58 | 32 | 67 |
| **1** | | | | | | | | |
| **0.6** | | | | | | | | |
| **0.3** | | | | | | | | |
| **0.15** | | | | | | | | |
| **0.075** | 4 | 7 | 4 | 8 | 4 | 10 | 4 | 10 |

[f] Published in Transportation Research Circular Number E-C044, October 2002

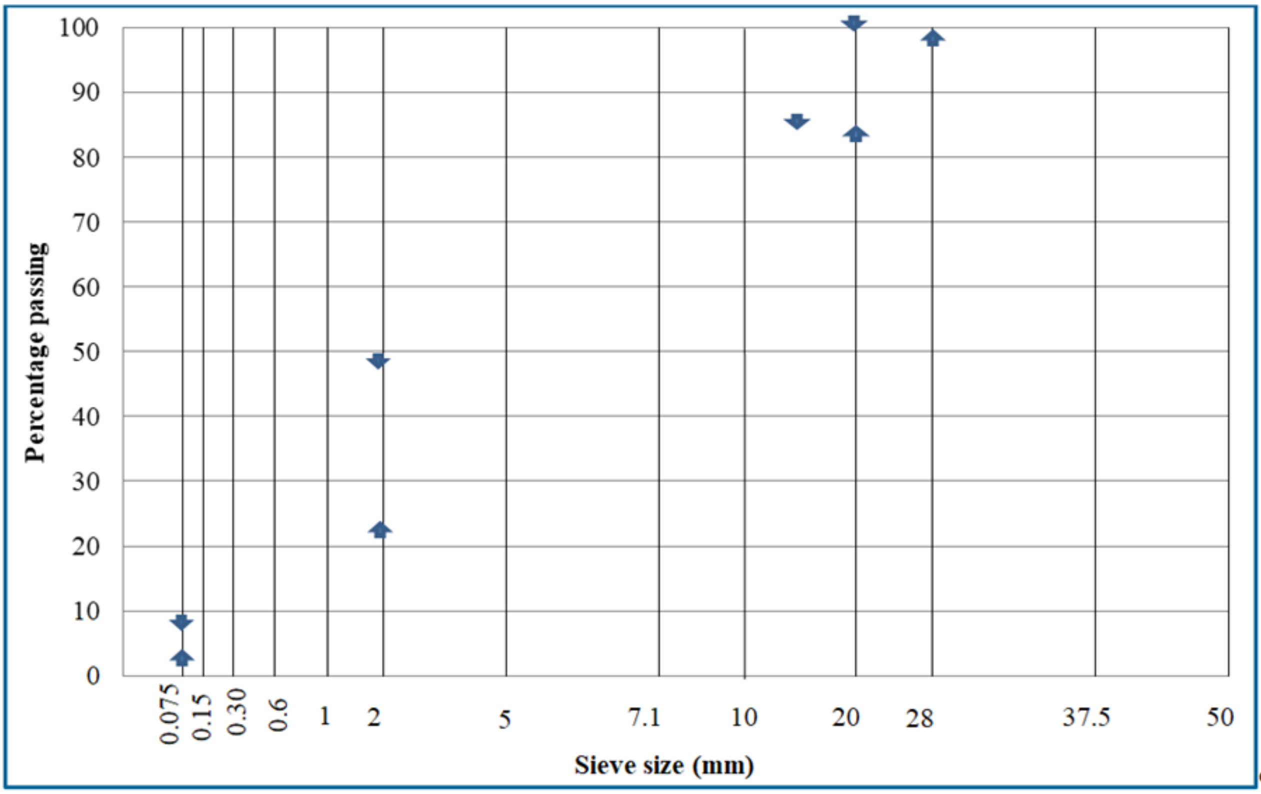


Figure 3: Grading control points plotted on 0.45 power chart for NMPS = 20 mm

### 4.6.2 Primary control sieves

The primary control sieve (PCS) controls the designation between coarse and fine aggregates. An aggregate grading that passes above the PCS control point is classified as fine-graded, whereas gradings passing below is classified as coarse-graded. Table 13 shows the percent passing control points of differentiation between coarse and fine mixes for various primary control sieves.

Table 13: Percent passing PCS control sieve

| NMPS | PCS | PCS control point [% passing] |
|---|---|---|
| **28 mm** | 7,1 mm | 40% |
| **20 mm** | 5 mm | 47% |
| **14 mm** | 2 mm | 39% |
| **10 mm** | 2 mm | 47% |

## 4.7 General requirements and specifications for aggregates

The following requirements are generally applicable to aggregates for asphalt:

- Coarse and fine aggregates obtained from crushing or natural sources should be clean and free from decomposed materials, vegetable matter and other deleterious substances;
- The aggregate blend may contain natural fines not obtained from the parent rock being crushed, subject to limitations of the proportion of such materials based on mix type and experience with the materials;
- The coarse aggregate is in most cases, crushed rock. Certain types of crushed blast-furnace slag may also be used, provided they satisfy the strength requirements and are not too water absorbent;
- The fine aggregate may be crusher sand, slag sand, clean natural sand, mine sand, selected river gravel or a mixture of these.

The standard test methods and recommended criteria to determine the suitability of aggregates for asphalt mix design are presented in Table 14.

## 4.8 Preparation and selection of aggregate grading

Steps and guidelines to obtain the design grading are as follows:

- Source samples of raw aggregate materials from stockpiles at asphalt plants as per TMH 5 C5. Each stockpile usually contains a given size of an aggregate fraction. A minimum of three fractions are used to generate a combined grading for the mix. These aggregates must be clean and free from decomposed materials, vegetable matter and other deleterious substances.
- Oven dry aggregates for a minimum of 16 hours at approximately 105°C. Samples for sieve analysis are reduced by (riffling / quartering). Ensure homogeneity of samples by mixing together, bags of similar aggregate sizes.
- Conduct wet sieve analysis test (SANS 3001-AG1) on randomly selected bags of samples to check if aggregates are adequately riffled. Determine the bulk and apparent densities for each coarse and fine aggregate fraction as per SANS 3001-AG20 and 3001-AG21, respectively. Also determine the bulk density of the mineral fillers as per BS 812 procedures.
- Determine properties of individual aggregate fractions. The recommended test methods and criteria are presented in Table 14.
- Combine the individual aggregate fractions into trial blends of a single grading by using a basic formula presented in Equation 4.1. Blends can be obtained by trial and error using Excel Solver or any commercially available software that does aggregate blending.

  $P = Aa + Bb + Cc, \ldots$ *Eq. 4.1)*

  $P$ = percentage of materials passing a given sieve for the combined aggregates *A, B, C …*

  *A, B, C….* = percentage of materials passing a given sieve for aggregates *A, B, C…*

  *a, b, c,….* = proportions (decimal fractions) of aggregates *A, B, C*, … in the blend (*a, b, c,….* = 1.00).

- Prepare a minimum of three trial aggregate blends; plot the grading of each trial blend on a 0.45-power chart, and, ***for sand skeleton mix types***, compare the gradings of the trial blends with the guidelines provided in Table 12 (i.e. control points for the design NMPS). In a situation where blended aggregate fails to meet these criteria, consideration should be given to adjusting the aggregate proportions.

**Table 14: Recommended tests and criteria for aggregate selection**

| Property | Test | Standard | Criteria |
|---|---|---|---|
| **Hardness / Toughness** | Fines aggregate crushing test: 10% FACT | SANS 3001-AG10 | Sand skeleton mixes: ≥ 160 kN<br>Stone skeleton mixes: ≥ 210 kN |
| | Aggregate crushing value (ACV) | SANS 3001-AG10 | Sand skeleton mixes ≤ 25<br>Stone skeleton mixes ≤ 21<br>Rolled in chippings ≤ 21 |
| **Soundness** | Magnesium sulphate soundness | SANS 5839<br>SANS 3001-AG12 | 12% to 20% is normally acceptable. Some specifications require ≤ 12% loss after 5 cycles |
| **Durability** | Methylene blue adsorption indicator | SANS 6243 | High quality filler: ≤ 5<br>> 5: additional testing and analysis needed |

| | | | |
|---|---|---|---|
| **Particle shape and texture** | Flakiness index | SANS 3001- AG4 | 20 mm and 14 mm aggregate: ≤ 25[g]<br>10 mm and 7.1 mm aggregate: ≤ 30<br>Rolled in chippings ≤ 20 |
| | Polished stone value (PSV) | SANS 3001–AG11 | Minimum 50[h] |
| | Fractured faces | ASTM 5821 | Sand skeleton mixes: at least 50% of all particles should have three fractured faces<br>Stone skeleton mixes & rolled in chippings: at least 95% of all particles should have three fractured faces |
| **Water absorption** | Coarse aggregate (> 5mm) | SANS 3001-AG20 | ≤ 1% by mass |
| | Fine aggregate (< 5mm) | SANS 3001- AG21 | ≤ 1.5% by mass |
| **Binder absorption** | Coarse and fine aggregate | SANS 3001-AS11 | ≤ 0.5% by mass |
| **Cleanliness** | Sand equivalency test | SANS 3001-AG5 | ≥ 50 total fines fraction |
| | Clay lumps and friable particles | ASTM C142–97 | ≤ 1% |

## 4.9 Surface area of aggregate

The surface area of the blended aggregate is important for the determination of binder content in the asphalt mix. The finer the mix grading, the larger the total surface area of the aggregate and the greater the amount of binder required to uniformly coat the aggregate particles. The *specific surface area* (*SA*) of the aggregate particles (in m$^2$/kg) is calculated based on Eq. 4.2:

$$SA = (2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.30e + 0.60f + 1.6g) \times 0.2048 \qquad \text{(Eq. 4.2)}$$

where:

*a* = percentage passing 5 mm sieve;
*b* = percentage passing 2 mm sieve;
*c* = percentage passing 1 mm sieve;
*d* = percentage passing 0.60 mm sieve;
*e* = percentage passing 0.30 mm sieve;
*f* = percentage passing 0.15 mm sieve, and
*g* = percentage passing 0.075 mm sieve

This calculation is based on a bulk density of the –5 mm fraction of the total aggregate of 2 650 kg/m$^3$. Consequently, in cases where the bulk density of this aggregate fraction has a different value, a correction is required as follows:

$$SA_N = SA \times \frac{2\,650}{BD_5} \qquad \text{(Eq. 4.3)}$$

where:

$SA_N$ = Normalised specific surface area
$BD_5$ = Bulk density of the aggregate fraction passing the 5 mm sieve, kg/m$^3$

[g] For certain types of mixes, e.g. UTFC and SMA a maximum flakiness index of 20 is preferred
[h] Consideration can be given to adopting a limiting value of 45, with due regard to material availability, traffic, road geometry and climate

# 5. Mix design

## 5.1 Introduction

The primary objective of asphalt mix design is to achieve a durable mix meeting certain specification criteria using an economical blend of aggregates and binder. To achieve this objective, the following are important performance factors to consider:

- Sufficient workability;
- Durability by having sufficient binder;
- Sufficient stability under traffic loads;
- Sufficient capacity for load transfer to underlying layers;
- Meeting volumetric criteria, and
- Resistance to moisture damage, permanent (plastic) deformation, and fatigue cracking.

The process of asphalt mix design involves the selection and blending of component materials, preparing compacted specimens, testing and evaluation of the optimum mix.

## 5.2 Asphalt mix properties

The main properties which are considered in the mix design are:

### 5.2.1 Workability

Workability is the ease of handling, placing and compacting the mix under the prevailing conditions. A number of factors that affect workability are:

- Mixes containing high percentage of coarse aggregates have the tendency to segregate and could present difficulties to attain a uniformly well compacted layer;
- Too high or too low filler in the mix;
- Too low or too high temperature will make the mix unworkable or tender, respectively;
- Excessive proportion of large sized aggregate in relation to the layer thickness (see section 2.2.5 Layer or lift thickness and maximum particle size)

For a given aggregate grading, workability can be improved by:

- Increase in binder content;
- Decrease in binder viscosity;
- Less angular aggregate;
- Limiting the maximum particle size to less than a third of the layer thickness;
- Construction controls that ensure the mix is compacted at the proper temperatures.

### 5.2.2 Durability

Durability of asphalt mix is its ability to resist:

- Hardening of the binder due to:
  - Oxidation;
  - Loss of volatiles;
  - Physical (steric) hardening;
  - Loss of oily substances due to absorption into porous aggregates (exudative hardening).
- Disintegration of the aggregate;
- Stripping of the bituminous binder from the aggregate;
- Action of traffic.

Durability of mixes can be improved by using:

- An appropriate binder in relatively thick films;
- Dense aggregate packing, i.e. low air voids;
- Sound, durable and strip resistant aggregates;
- Use of adhesion-promoting or anti-stripping additives or hydrated lime.

### 5.2.3 Stiffness

The stiffness of asphalt determines its ability to carry and spread traffic loads to underlying layers. Relatively stiff asphalt is generally required for asphalt bases. Less well supported surfacing layers e.g. pavement structures with a lower radius of curvature associated with higher vertical deflection, may be better served by a lower stiffness asphalt, to avoid traffic induced cracking, provided the underlying support is still adequate to carry the traffic loads. The stiffness of asphalt is mostly influenced by:

- Transient traffic loading time;
- Temperature;
- Binder content and binder rheology;
- Aggregate packing;
- Degree of compaction achieved during construction.

### 5.2.4 Resistance to permanent deformation (Rutting)

The ability of an asphalt mix to resist permanent or plastic deformation under the influence of traffic and elevated temperatures depends primarily on:

- Internal frictional resistance of the aggregates in the mix;
- Cohesion (tensile strength) resulting from the bonding ability of the binder in the mix;
- Cohesive strength, i.e. resistance to viscous flow of the binder at elevated temperatures.

Rutting can typically occur during the summer pavement temperatures in excess of 45°C which frequently occur in South Africa in summer. Under such conditions deformation is resisted by the frictional resistance in the aggregate and binder stiffness. The predominant factor would be dependent on the mix type, e.g. stone or sand skeleton.

### 5.2.5 Resistance to fatigue cracking

Resistance to fatigue cracking is the ability of the mix to withstand repeated tensile strains without fracture. Fatigue failure in asphalt layers occurs when the number of repetitions of applied loads exceeds the capacity of the asphalt to withstand the associated tensile strains. The situation may be worsened by stresses induced by thermal fluctuations.

The rheological properties of the bituminous binder in the asphalt play a key role in the capacity of the asphalt to resist fatigue. On the one hand, accelerated binder ageing due to high voids, which or low binder content could lead to low fatigue life. On the other hand, bituminous binders with good stress relaxation properties or operating at elevated temperatures, would enhance the resistance of the layer to premature fatigue distress.

Generally thin asphalt layers are more prone to fatigue as a result of high deflections or bending when compared with thick asphalt layers.

### 5.2.6 Permeability

Permeability of asphalt is a measure of the penetration of the mix by air, water and water vapour. Low permeability of a dense asphalt surfacing promotes long term durability and protects underlying layers from the ingress of water, which may lead to failure. Since asphalt layers in South Africa – particularly wearing courses – are relatively thin (typically 40 mm), permeability is a critical factor particularly where, as is often the case, continuously graded wearing courses overlie granular bases which are sensitive to the ingress of water. Another issue is the potential for binder hardening and stripping in mixes with high permeability.

Factors that reduce permeability are:

- High binder contents with adequate film thickness;
- Dense aggregate packing;
- Dispersed rather than inter-connected air voids within the mix, such as are found in gap-graded mixes;
- Well compacted asphalt layers.

Since water permeability tests are often problematic in that high variability can be experienced given the complexity of flow paths for a particular testing location, air permeability is prescribed to quantify permeability in terms of experience gained.

### 5.2.7 Thermal fracture

Thermal fracture of asphalt can arise due to contraction and expansion of the asphalt layer under extreme temperature changes. The potential for low temperature cracking is an interplay between the environment, the road structure and, importantly, the properties of the asphalt mixture, including the binder. The performance grade specification for bituminous binders, currently being implemented on a trial basis will provide criteria which will safeguard against the use of binders that are not unduly susceptible to thermal cracking.

## 5.3 Composition of asphalt

Asphalt is composed of aggregate, mineral filler, bituminous binder, and frequently reclaimed asphalt. The design of asphalt mixes entails largely the process of selecting and proportioning these materials to obtain the desired properties in the final product.

Procedures and criteria for selecting the component materials for asphalt mixes were presented in Chapter 3 and Chapter 4.

## 5.4 Mass and volumetric concepts and definitions

Various *mass* and *volumetric* concepts, commonly used in the design of asphalt, are illustrated in the schematic representation of compacted asphalt mix shown in Figure 4.

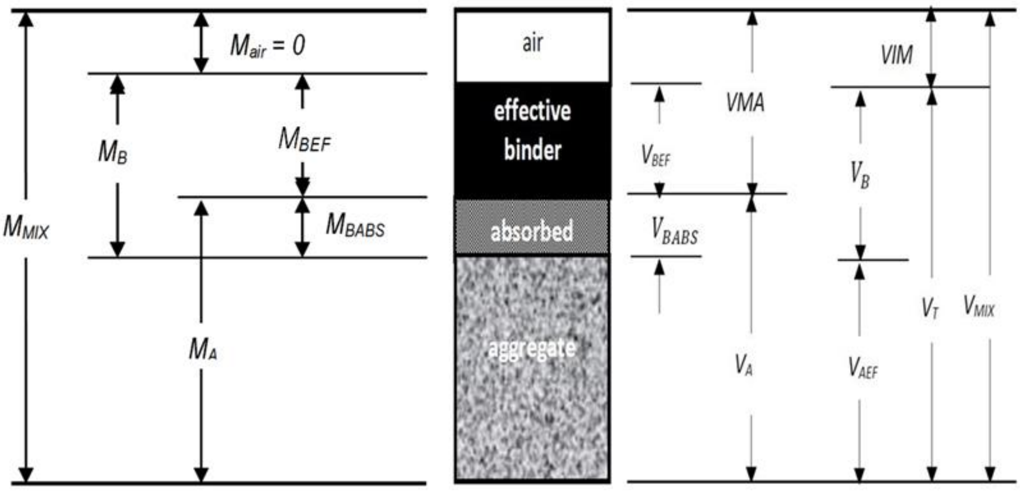


Figure 4: Volumetric parameters of a compacted asphalt specimen

Both the mass and volume parameters in the above illustration are defined in

Table 15.

Table 15: Mass and volume parameter definitions

| Mass concepts | | Volume Concepts | |
|---|---|---|---|
| $M_{MIX}$ | Oven dry mass of mix specimen | $V_{MIX}$ | Volume of mix specimen |
| $M_{air}$ | Mass of air = 0 | $VIM$ | Percentage voids in the mix |
| $M_B$ | Mass of total binder | $V_B$ | Volume of binder in the mix |
| $M_{BABS}$ | Mass of binder absorbed into aggregate | $V_{BABS}$ | Volume of binder absorbed into aggregate |
| $M_{BEF}$ | Mass of effective binder | $V_{BEF}$ | Volume of effective binder i.e. that which does not penetrate into aggregate pores |
| $M_A$ | Mass of dry aggregate | $V_A$ | Volume of aggregate |
| | | $VMA$ | Volume of voids in the mineral aggregate |
| | | $V_T$ | Total volume of aggregate and binder |
| | | $V_{AEF}$ | Volume of aggregate excluding pores with absorbed binder |

Table 16 lists the definitions of the density parameters and associated test standards used routinely in the design of asphalt mixes. The definitions of density parameters for aggregate are illustrated in Figure 5 (on page 50).

Table 16: Density parameters used in volumetric analysis

| Parameter | Symbol | Definition | Method |
|---|---|---|---|
| Bulk density of aggregate | $BD_A$ | Mass of the aggregate particles divided by the volume of the aggregate particles including the impermeable (internal), and permeable (surface) pores, but excluding the inter-particle voids, expressed in kilograms per cubic metre (kg/m$^3$) | SANS 3001-AG20 (> 5 mm)<br>SANS 3001-AG21 (< 5 mm) |
| Apparent density of aggregate | $AD_A$ | Mass of the aggregate particles divided by the volume of the aggregate particles including impermeable (internal) voids but excluding any pores or capillaries that become filled with water after extensive[i] soaking, and inter-particle voids, expressed in kilograms per cubic metre (kg/m$^3$) | SANS 3001-AG20 (> 5 mm )<br>SANS 3001-AG21 (< 5 mm ) |
| Effective density of aggregate | $BD_{AEF}$ | Mass of the aggregate particles divided by the volume of the aggregate particles excluding the volume of pores that absorb binder and inter-particle voids, expressed in kilograms per cubic metre (kg/m$^3$) | |
| Water absorption | $W_{ABS}$ | Difference in mass between the saturated surface-dry condition and the oven-dry condition of a given volume of aggregate | SANS 3001-AG20 (> 5 mm)<br>SANS 3001-AG21 (< 5 mm) |
| Bulk density of binder | $BD_B$ | The bulk density of the binder, expressed in kilograms per cubic metre (kg/m$^3$) | ASTM D70 |

[i] 15 – 19 hour duration

| | | | |
|---|---|---|---|
| Bulk density of mix[j] | $BD_{MIX}$ | Mass per unit volume, including the air voids, of a bituminous mixture at a known test temperature, expressed in kilograms per cubic metre (kg/m³) | Sabita Manual 39: ASP8 |
| Maximum voidless density of the mix (Rice method) | $MVD$ | Mass per unit volume of a voidless bituminous mixture at a known test temperature, expressed in kilograms per cubic metre (kg/m³) | Sabita Manual 39: ASP9 |
| Coarse aggregate loose unit weight | LUW | The loose unit weight of an aggregate is the amount of aggregate that fills a unit volume without any compactive effort applied. | AASHTO T-19 |
| Coarse aggregate rodded unit weight | RUW | The rodded unit weight of aggregate is the amount of aggregate that fills a unit volume with compactive effort applied | AASHTO T-19 |

**Note 5.1:** For the purpose of calculations, the bulk density of penetration grade binder may be taken as 1.020 kg/m³. However, the density of bitumen may vary significantly from this figure and the designer should approach the supplier to ensure that the value adopted for design purposes is relevant to the bitumen that will be used in the project. Where modified binders are used the value of the bulk density of the binder should be obtained from the supplier (SANS 3001-AS11).

## 5.5 Determination of mix design parameters

### 5.5.1 Bulk density of the mix

For both design and judgement of compliance (acceptance control) purposes, the volume associated with bulk density of asphalt is determined either by the vacuum sealing (VS) method (AASHTO T 331) or by measurement of dimensions, both as described in Sabita Manual 39: protocol ASP8. The criteria for the selection of the appropriate procedure for testing are as follows:

- For dense mixtures, other than those purposely designed to be porous (i.e. with void content in excess of 12%) – use the vacuum sealing method as described in AASHTO T 331
- For mixtures purposely designed to have an open surface and high porosity, with void content > 12 %, by direct dimensional measurement

In the AASHTO T 331 standard the bulk *specific gravity* (*Gmb*) of the specimen is determined. The bulk density of the specimen $BD_{MIX}$, in kilogram per cubic metre (kg/m³), is calculated as follows:

$$BD_{MIX} = Gmb\rho_W \qquad \text{Eq. (5.1)}$$

where:

*Gmb* is the bulk specific gravity of the specimen;

$\rho_W$ = density of water at 25°C, (997.1 kg/m³).

In cases where the *volume of the specimen is determined by measurement of dimensions*, the following expressions apply:

$$V_{MIX} = l \times b \times h \qquad \text{Eq. (5.2a)}$$

or

[j] See note 5.2

$$V_{MIX} = \left(\frac{\pi \times diameter^2}{4}\right) \times h \qquad \text{Eq. (5.2b)}$$

where:

l, b, h and diameter are specimen dimensions, *expressed in millimetres (mm)*

The *Bulk Density of the mix specimen*, in kilograms per cubic meter (kg/m3) is calculated as:

$$BD_{MIX} = \frac{10^6 \times M_4}{V_{MIX}} \qquad \text{Eq. (5.3)}$$

where:

$M_4$ is the oven dry mass of the mix specimen, expressed in grams.

In such cases, the *Bulk Density of the mix specimen* ($BD_{MIX}$), expressed, in kilograms per cubic metre (kg/m$^3$), is determined using the following formula:

$$BD_{MIX} = \left(\frac{M_4}{M_2 - M_3}\right) \rho_W \qquad \text{Eq. (5.4)}$$

where:

$M_2$ is the saturated surface-dry mass of the specimen, expressed in grams (g);

$M_3$ is the mass of the specimen in water, expressed in grams (g);

$M_4$ is the oven dry mass of the specimen, expressed in grams (g); and

$\rho_w$ is the density of water at 25°C: 997.1 kg/m$^3$ (to be corrected as per Table 1 in SANS 3001-AS10 if measurements are determined at another test temperature.

### 5.5.2 Maximum Voidless Density of the Mix

The *Maximum Voidless Density (MVD*) of asphalt mixes and the quantity of binder absorbed by the aggregate are determined according to Sabita Manual 39 protocol ASP9. The mass definitions are:

*M1* is the mass of the flask assembly, expressed in grams (g);

*M2* is the mass of the flask assembly and sample, expressed in grams (g);

*M3* is the mass of the flask assembly and sample filled with water, expressed in grams (g);

*M4* is the mass of the flask assembly filled with water, expressed in grams (g); and

*M5* is the mass of the aggregate surface dry in air, expressed in grams (g); and

$\rho_W$ is the density of water at the test temperature in kilograms per cubic meter (kg/m$^3$)

The *Maximum Voidless Density* (*MVD*) of the mix, expressed in kilograms per cubic metre (kg/m$^3$), is calculated to the nearest 1 kg/m$^3$ as follows:

*(a) When no water is absorbed by the coated aggregate:*

$$MVD = \left\{\frac{(M_2 - M_1)}{\frac{((M_4 - M_1) - (M_3 - M_2))}{\rho_W}}\right\} \qquad \text{Eq. (5.5a)}$$

*(b)* *When water is absorbed by the coated aggregate[k]:*

$$MVD = \left\{\frac{(M_2 - M_1)}{\frac{(M_4 - M_3 + M_5)}{\rho_W}}\right\} \qquad \text{Eq. (5.5b)}$$

### 5.5.3 Notes on density measurements

As indicated above the three generally accepted types of density for aggregate use in asphalt are the following:

- Apparent density ($A_{DA}$);
- Bulk (dry) density ($BD_A$); and
- Effective density ($BD_{AEF}$).

*Apparent density* considers the volume of the aggregate itself. It does not include the volume of any pores or capillaries that become filled with water after extensive soaking. It does however include pores that are not accessible to water penetration.

*Bulk (dry) density* considers the overall volume of the aggregate particle, including impermeable pores and the pores that become filled with water after soaking.

*Effective density* considers the overall volume of the aggregate excluding the volume of pores that absorb binder.

Whereas bulk and apparent densities can relate to individual aggregates or combined aggregates, effective density relates exclusively to the total combined aggregate structure in a mix.

These volumetric concepts (denominators in density expressions) related to aggregates are illustrated conceptually in Figure 2. The volumes associated with the determination of densities listed are indicated with blue outlines.

Since $AD_A$, $BD_A$ and $BD_{AEF}$ all have the same mass as numerator of the density expression and only differ in the volume as denominator, the following inequality is always true.

$$AD_A \geq BD_{AEF} \geq BD_A$$

Note: The volumes will only ever be equal for non-absorptive aggregate.

[k]) In cases where the aggregate is not properly sealed by the binder are rare in SA. Where absorption may have occurred the aggregate should be inspected as described in Sabita Manual 39: protocol ASP9 .

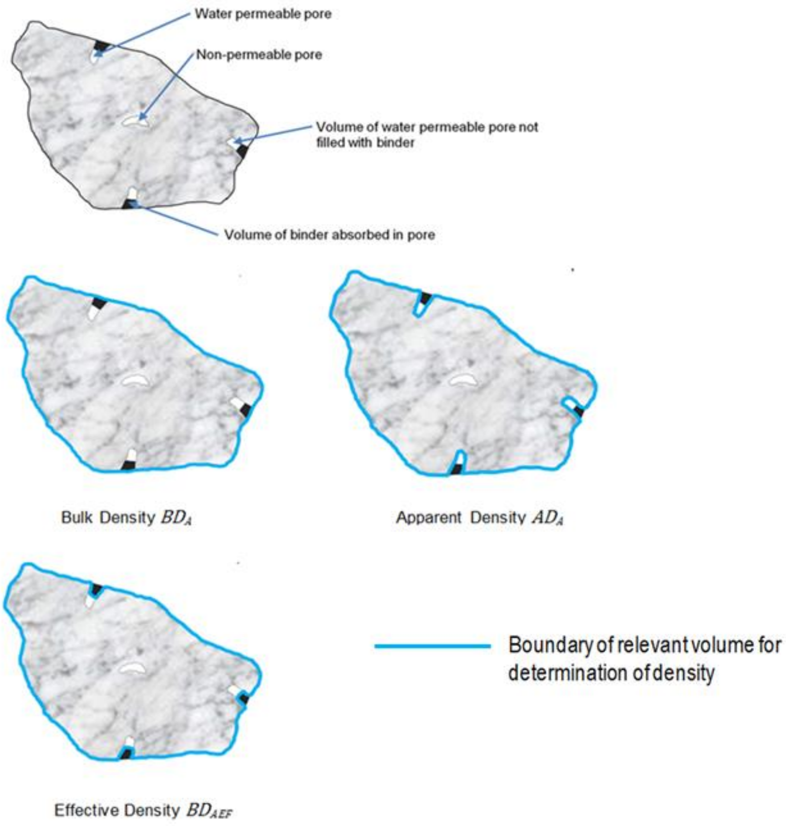


**Figure 5: Conceptual illustration of the volumes associated with the determination of Bulk, Apparent & Effective Density of aggregates**

### 5.5.3 Additional volumetric calculations

Other compositional parameters and their definitions used routinely in the design of asphalt are listed in Table 17.

Table 17: Compositional parameters used in volumetric analysis

| Parameter | Symbol | Definition | Formula |
|---|---|---|---|
| Binder mass | $M_B$ | Mass of binder in the mix, expressed in grams (g) | $M_B = B\left(\frac{M_2-M_1}{100}\right)$ Eq. (5.6)<br>where B is the percentage binder in the mix |
| Aggregate mass | $M_A$ | The mass of the aggregate in the mix, expressed in grams (g) | $M_A = \frac{(100-B)(M_2-M_1)}{100}$ Eq. (5.7) |
| Binder volume | $V_B$ | Volume of binder in the mix, expressed in cubic centimetres (cm³) | $V_B = \frac{1000\times M_B}{BD_B}$ Eq. (5.8)<br>where $BD_B$ is the bulk density of the binder, in kg/m³ |
| Aggregate volume | $V_A$ | Volume of the aggregate in the mix, expressed in cubic centimetres (cm³) | $V_A = \frac{1000\times M_A}{BD_A}$ Eq. (5.9)<br>where $BD_A$ is the bulk density of the aggregate in kg/m3 |
| Binder content | $B$ | Proportion of binder, expressed as a percentage of total mix | $B = 100\times\left(\frac{M_B}{M_A+M_B}\right)$ Eq. (5.10) |
| Mix volumes | $V_T$ | Total volume of aggregate and binder in the mix in cm³ | $V_T = V_A + V_B - V_{BABS}$ Eq. (5.11) |
| | $V_{DA}$ | De-aired volume of the mix in cm³ | $V_{DA} = \frac{1000}{\rho W}\times(M_4 + M_3 - M_5)$ Eq. (5.12) |
| Binder proportions | $V_{BABS}$ | Volume of binder absorbed into the pores (permeable voids) in the aggregate) | $V_{BABS} = V_A + V_B - V_{DA}$ Eq. (5.13) |
| | $M_{BABS}$ | Mass of the binder absorbed in the mix, expressed in grams (g) | $M_{BABS} = \frac{(V_{BABS}\times BD_B)}{1000}$ Eq. (5.14) |
| | $B_{BABS}$ | Percentage of binder absorbed by the aggregate expressed as a percentage of the mass of the dry aggregate in the mix | $B_{ABS} = 100\times\left(\frac{M_{BABS}}{M_A}\right)$ Eq. (5.15) |
| | $M_{BEF}$ | The mass of effective binder in the mix, expressed in grams (g) | $M_{BEF} = M_B - M_{BABS}$ Eq. (5.16) |
| | $B_{EF}$ | The percentage of effective binder in the mix, expressed as a percentage of the mass of the mix | $B_{EF} = \frac{100\times M_{BEF}}{M_A+M_B}$ Eq. (5.17) |
| Voids in the mix | $VIM$ | Difference between the *MVD* and the $BD_{MIX}$, expressed as a percentage of the *MVD* | $VIM = 100\times\left[\frac{(MVD-BD_{Mix})}{MVD}\right]$ Eq. (5.18) |
| Voids in the mineral aggregate | $VMA$ | Volume of voids in the bulk mix expressed as the % difference between the volume of aggregate and the bulk volume of the mix | $VMA = 100 - \left(\frac{(100-B)BD_{MIX}}{BD_A}\right)$ Eq. (5.19a)<br>Alternatively, if the percentage voids in the mix, *VIM*, and the volume of effective binder, expressed as a percentage of the mix volume, $B_{EF}$, is known:<br>$VMA = VIM + V_{BEF}$ Eq. (5.19b) |
| Effective binder volume | $V_{BEF}$ | Volume of effective binder expressed as a percentage of the volume of the bulk mix | $V_{BEF} = \frac{B_{EF}\times BD_{MIX}}{BD_B}$ Eq. (5.20a)<br>Alternatively, if VMA and VIM are known:<br>$V_{BEF} = VMA - VIM$ Eq. (5.20b) |
| Voids filled with binder | $VFB$ | Percentage of voids in the bulk mix filled with effective binder | $VFB = \frac{100\times V_{BEF}}{VMA}$ Eq. (5.21) |
| Filler-binder ratio | FBR | Ratio of the mass of filler to the mass of binder | $FBR = \frac{Mass\ of\ filler}{Mass\ of\ binder}$ Eq. (5.22) |

## 5.6 Mix design levels

This manual presents four levels of mix design i.e., Level IA & B, Level II, and Level III. The use of levels allows for the selection of a design process that is appropriate for the traffic loads and volumes (expressed as E80s) over the service life of the asphalt pavement and mitigation of exposure to the risks associated with structural damage.

In terms of the traffic classification given in Table 1, the mix design levels being adopted in terms of the desired risk profile are shown in Table 18.

Table 18: Mix design levels for traffic volumes over service life of the pavement

| Design traffic [E80 | Description | Mix design level |
|---|---|---|
| **< 0.3 million** | Low / Light | Level IA |
| **0.3 to 3 million** | Medium | Level IB |
| **>3 to 30 million** | Heavy | Level II |
| **> 30 million** | Very heavy – to Extreme | Level III |

Figure 6 presents general recommendations for applying the three design levels.

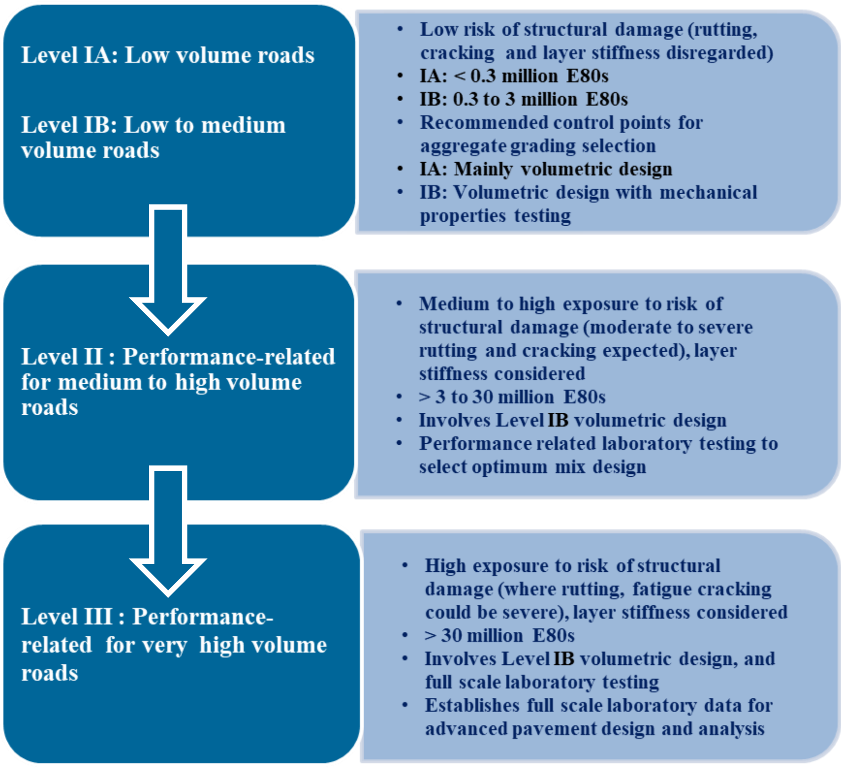


Figure 6: Mix design levels

### 5.6.1 Level I mix design process

The design process for Level I is shown in Figure 7.

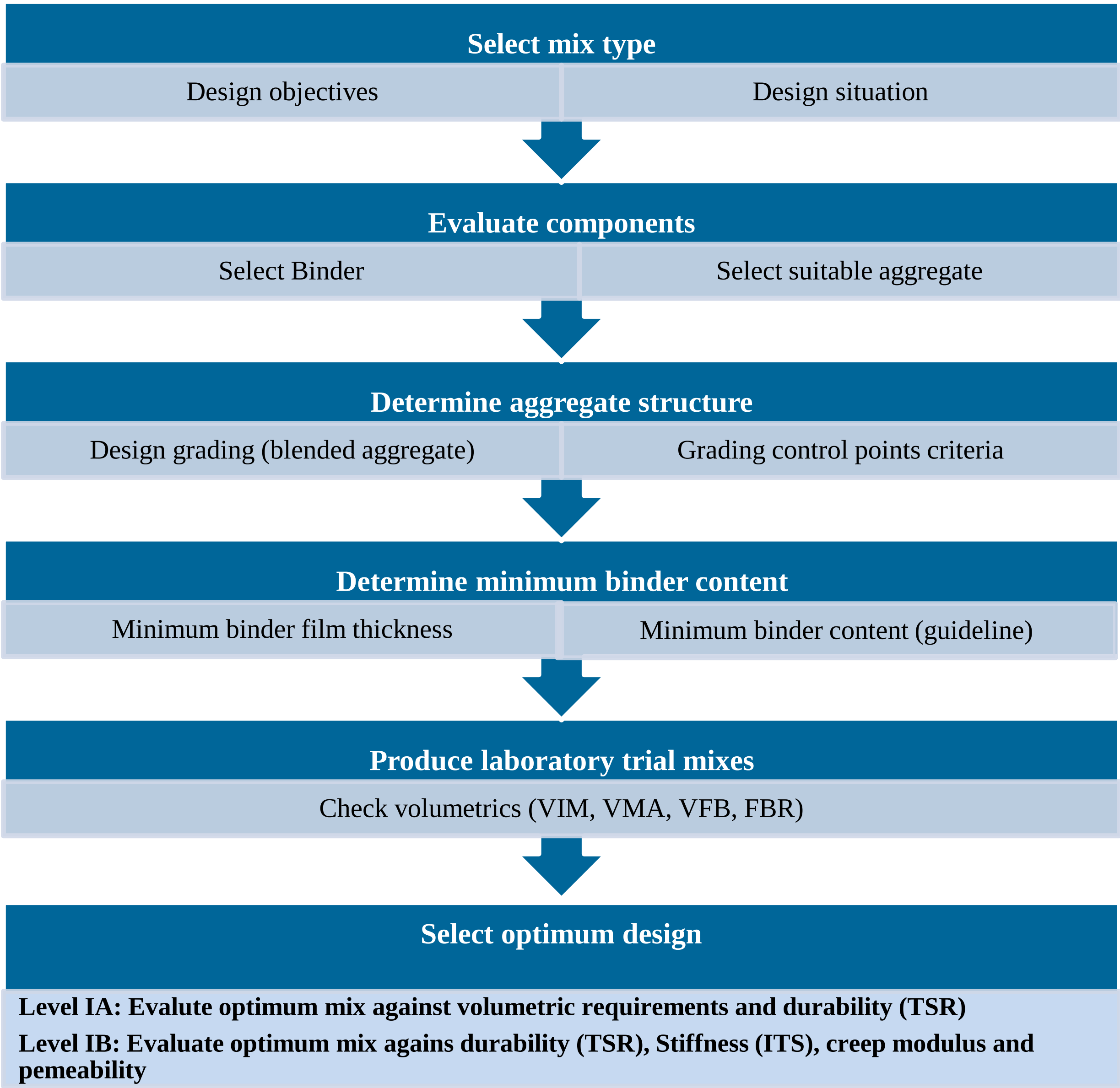


Figure 7: Level I design process

The basic steps involved in the Level I mix design are as follows:

(1) Select mix type based on design objective and situation (see Chapter 2)
(2) Select a binder that is appropriate for the climate and traffic situation at the project site. Once available, the selection of an appropriate performance grade (PG) binder is recommended.
(3) Select aggregates – the aggregates must meet all specification requirements of the project. The procedures and acceptance requirements described in Chapter 4 should be followed to select aggregate fractions for the mix design.
(4) Develop three trial aggregate blends (gradings) from the selected aggregate fractions. The design aggregate composition is established by:
    (a) Determine minimum binder content for each trial blend using the minimum requirements for binder film thickness, specific surface area and density of the –5 mm fraction of the aggregate blend. In calculating the binder film thickness, the designer should note that the volume of binder used is the ***effective binder***, i.e. the volume of binder NOT absorbed by the aggregate. Eq. 5.1 yields the film thickness, $F$, in µm (micron):

$$F = \frac{B_{BEF}}{(100-B)} \cdot \frac{1}{SA_N} \cdot \frac{1000}{BD_B} \qquad \text{Eq. 5.22}$$

where:

$B_{BEF}$ is the *Effective binder content* expressed as a percentage of the total mass of the mix according to Eq. 5.17

$B$ *is the Total binder content* expressed as a percentage of the total mass of the mix.

$BD_B$ is the density of the binder at 25°C

$SA_N$ is the *Normalised surface area* calculated as per Eq. 4.3

***The total binder content of the mix should be such that the binder film thickness, $F$, based on the effective binder content, shall be ≥ 5.5 µm.***

**Note 5.3** The determination of the specific surface area according to Eq. 4.2 above yields a ***theoretical value*** and, given the wide variety of aggregate shapes and textures, is not a precise computation of the actual area. Its value lies in being a ***consistent comparative parameter.***

**Note 5.4** In determining the minimum binder content, the requirements in respect of the filler / binder ratio should be given due consideration.

**Note 5.5** Use of the binder film thickness, $F$, is recommended practice for asphalt in general use, i.e. sand and stone skeleton mix types. When designing EME, the determination of minimum binder content to satisfy a *Richness Modulus* requirement is the correct practice, as per Sabita Manual 33. Additionally the binder content of SMA should be designed in accordance with Appendix B.

(b) Determine an optimum asphalt mix design for this level i.e. Levels IA and IB

**<u>For Level IA mix design</u>**

Steps to select the optimum mix for this level of design are as follows:

i. Select four trial binder contents based on; (1) minimum binder content, (2) minimum binder content + 0.5%, (3) minimum binder content + 1.0%, and (4) minimum binder content + 1.5% by mass of total mix.
ii. Determine filler-binder ratio
iii. Prepare[I] three duplicate mixes for each trial binder content. Each trial binder content should be mixed with the same design aggregate composition. Also prepare two loose asphalt samples specimens to determine the maximum void-less density (MVD) as per SANS 3001-AS11.
iv. Specimens should then be short-term aged by placing the loose mix in an oven at 135°C for 4 hours regardless of the aggregate absorption. Check that the sample temperature does not go below the compaction temperature.
v. Samples should be mixed and compacted at the appropriate mixing and compaction temperatures based on the selected binder type or grade. Mixing temperature is the range of temperatures that yields a binder viscosity (rotational) of approximately 0.17 ± 0.02 Pa.s, whereas the compaction temperature is obtained at viscosity of 0.28 ± 0.03 Pa.s. The appropriate temperatures for mixing and compaction should be based on temperature / viscosity information provided by the supplier of the bituminous binder in use.
vi. Compact the trial specimen (102 mm in diameter by 64 mm in height) for each trial binder content. The standard Marshall method for making asphalt briquettes (as contained in SANS 3001-AS1) should be followed, except that the recommended number of blows to compact the specimens is given in Table 19.

[I] Each aggregate fraction should be riffled down to as close as possible to the mass required. Scooping from a crate should not be allowed, as this leads to variance between individual trial blends

Table 19: Compaction requirements

| Design traffic [E80s] | No. of blows |
|---|---|
| **< 0.3 million** | 75 /45[m] |

vii. Determine the bulk density (BD) of the compacted samples as set out in section 0
viii. Use the BD and MVD results (average values for each trial binder content) to obtain the following volumetric parameters of the mix:
- Voids in the total mix (VIM).
- Voids in the mineral aggregate (VMA).
- Voids filled with binder (VFB).

ix. Design criteria – the criteria presented in Table 20 and Table 22 must be met for the mix.

**Note 5.6** The values of VMA in Table 21 are given to guide the designer in determining the optimum composition of aggregate fractions of sand skeleton mixes so as to reasonably ensure that critical parameters such as binder film thickness, voids and durability requirements will be met. ***It should not be considered as a requirement for specification purposes.***
These values apply to ***continuously graded, sand skeleton mixes*** only. The volumetric requirements for stone skeleton mixes should be determined using the Bailey method, or, in the case of SMA, reference to Appendix B – see note 4.7.

Table 20: Voids requirements

| Property | < 0.3 million E80s | |
|---|---|---|
| | Minimum | Maximum |
| **VIM (%)** | 3 | 5 |

Table 21: Minimum percent VMA

| NMPS (mm) | Minimum VMA* |
|---|---|
| **28** | 10 |
| **20** | 11 |
| **14** | 12 |
| **10** | 13 |

Table 22: Percent VFB

| Minimum | Maximum |
|---|---|
| 70 | 80 |

x. Determine optimum binder content based on the combined results of Marshall stability and flow (SANS 3001-AS1), density and void analyses as follow:
- Draw graphs of the following six relationships:
  - Bulk density versus binder content.
  - Marshall Stability versus binder content.
  - Marshall Flow versus binder content.
  - Air voids versus binder content.
  - VMA versus binder content.

[m] 75 blows on the first side + 45 blows on the reverse side of the specimen

– VFB versus binder content.

- From the graph of air voids versus binder content, determine the binder content at 4%.
- From the graph of bulk density versus binder content, determine the binder content at maximum (peak) bulk density.
- From the graph of stability versus binder content, determine the binder content at maximum (peak) stability.
- The three binder contents selected at 4% air voids, maximum BD and stability are averaged to determine the optimum binder content.

xi. Mix acceptance – if one or more of the mix design criteria cannot be met, then consider adjustments to aggregate type, grading, and /or binder type in the design procedures.

**Note 5.7**: The sole purpose of the Marshall method in this design procedure is to determine the optimum binder content for Level IA design.

**Note 5.8**: Where required the durability of the optimum mix can be assessed (as in Level IB) by conducting the Modified Lottman testing (ASTM D4867M) on the mix. See Table 25 below.

**For Level IB mix design**

Steps to select the optimum mix for this level of design are as follows:

i. Steps (i) to (v) of the procedures for Level IA should be followed.

ii. For each trial blend, compact the three duplicate specimens (150 mm in diameter by 115 mm in height) in a Superpave gyratory compactor following the test procedures contained in AASHTO T 312. Also, prepare two loose asphalt samples for each trial mix for determination of the maximum void-less density (MVD) of the mix using SANS 3001-AS11. Compact specimens immediately after completion of short-term oven conditioning to $\mathrm{N}_{\mathrm{design}}$ (the number of gyrations at which the air voids content equal to 4 percent) in accordance with Table 23.

iii. Determine the bulk density (BD*)* of the compacted specimens in accordance with SANS 3001-AS10.[n] Use the BD and MVD results (average values for each trial binder content) to compute the volumetric properties (VIM, VMA, VFB) of the mix at $\mathrm{N}_{\mathrm{design}}$.

iv. Select the design aggregate grading and a corresponding minimum binder content on the basis of satisfactory conformance of a trial blend with requirements for VIM, VMA, and VFB at design compaction level $\mathrm{N}_{\mathrm{design}}$.

Table 23: Compaction requirements for Level IB

| Design traffic (E80s) | $N_{design}$ |
|---|---|
| **0.3 – 3 million** | 75 |

Use the selected design aggregate grading to determine the optimum mix. Steps to select the optimum mix for this level of design are as follows:

i. Use the volumetric data to generate graphs of VIM, VMA and VFB versus binder contents. The design (optimum) binder content is established at 4 percent air voids (on the VIM versus binder content graph). The VMA and VFB are checked at the design binder content to verify that they meet the criteria presented in Table 21 and Table 24.

ii. The durability of the optimum mix design is assessed by conducting the Modified Lottman testing (ASTM D4867M) on the mix. Prepare short-term aged loose samples, and compact the specimens to in-place voids (typically, 7% ± 0.5% for continuously graded mixes). A reasonable rule of thumb is that in-place voids are approximately equal to design

[n] See note 5.2

voids +3%. Calculate the tensile strength ratio, and check results against the criteria presented in Table 25.

iii. A summary of the requirements and criteria to attain the optimum design for Level IB are given in Table 26.

iv. Mix acceptance – if one or more of the mix design criteria cannot be met, then consider adjustments to be made in aggregate type, grading, or binder type in the design process.

Table 24: Percent VFB

| Minimum | Maximum |
|---|---|
| 65 | 75 |

(c) Notes on interrelationships of volumetric parameters

**Note 5.9:** High VMA in the dry aggregate creates more space for the binder. Increasing the density of the mix by changing the grading of the aggregate may result in low VMA values with thin films of binder leading to a low durability mix. Recommendations to increase VMA if a change in the design aggregate is required are:

- Reduce the amount of material passing 0.075 mm fraction, however if the dust content is already low, this is not a viable option;
- Reduce percentage of rounded natural sand and use a higher percentage of angular or crushed sand;
- Change the aggregates to incorporate material with better packing characteristics (e.g., fewer flaky aggregate particles). Use highly angular and a rougher surface texture aggregate particles.

**Note 5.10:** The effect of grading on VMA is somewhat complex; however denser gradings generally lead to a decrease in VMA. Also larger aggregates (NMPS) reduce VMA. Low VMA is very sensitive to slight changes in binder content. Generally, economising the binder content by lowering VMA is counter-productive and should be avoided.

**Note 5.11:** VFB restricts the allowable air void content for mixes which are near the minimum VMA criteria. Mixes designed for lower traffic volumes may not pass the VFB requirement with a relatively high percent air voids in the field even though the air void range requirement is met. Also, in mixes requiring high voids content and relatively thick binder films, e.g. open graded mixes and UTFC's the maximum limit of 75% will be exceeded. Meeting VFB requirements avoids less durable mixes resulting from thin films of binder on the aggregate particles.

**Note 5.12:** The lower limit of VFB range should always be met at 4 percent air voids if the VMA requirements are met. If the VFB upper limit is exceeded, then the VMA is substantially above the minimum required. In a situation like this, the mix should be re-designed to reduce the VMA in the interests of cost savings. The following options should be considered in such a situation:

- Increase the amount of material passing 0.075 mm fraction. The dust content should be increased if there is enough room available within acceptable limits;
- Change the aggregates to incorporate material with better packing characteristics (e.g., fewer flaky aggregate particles). Use highly angular and a rougher surface texture aggregates.

Table 25: Moisture resistance criteria (Minimum TSR)

| Layer type | |
|---|---|
| Base | Wearing course |
| 0.70 | 0.80 |

**Note 5.13:** If TSR is less than the specified values, then adjust the mix design to increase the moisture resistance of the mix to an acceptable level. Such adjustments may include adding hydrated lime to the mix, adding some type of liquid anti-strip additives, or changing the source of the aggregate or binder, or both.

Table 26: Summary of empirical performance tests for Level IB

| Property | Test | Method | Criteria |
|---|---|---|---|
| **Durability/TSR** | Modified Lottman | ASTM D 4867 M | See Table 25 |
| **Stiffness** | Indirect tensile strength | ASTM D 6931-07 | 900 kPa - 1 650 kPa @ 25°C |
| **Creep modulus** | Dynamic creep | CSIR RMT 004 | 10 MPa min. @ 40°C |
| **Permeability** | Air permeability | Sabita Manual 39: ASP5 | ≤ 1 X $10^{-8}$ $cm^2$ |

**Note 5.14:** Stone-skeleton mixes and mixes manufactured with some polymer modified or bitumen-rubber binders may have low dynamic creep values and still exhibit good resistance to rutting. This test may therefore not be applicable for such mixes.

## 5.6.2 Level II and Level III design process

The design process for Level II and Level III is shown in Figure 8. Compared to Level II, a complete set of laboratory data is collected at Level III to predict stiffness, permanent deformation and fatigue, the purpose being to establish a direct link between mix design and pavement design.

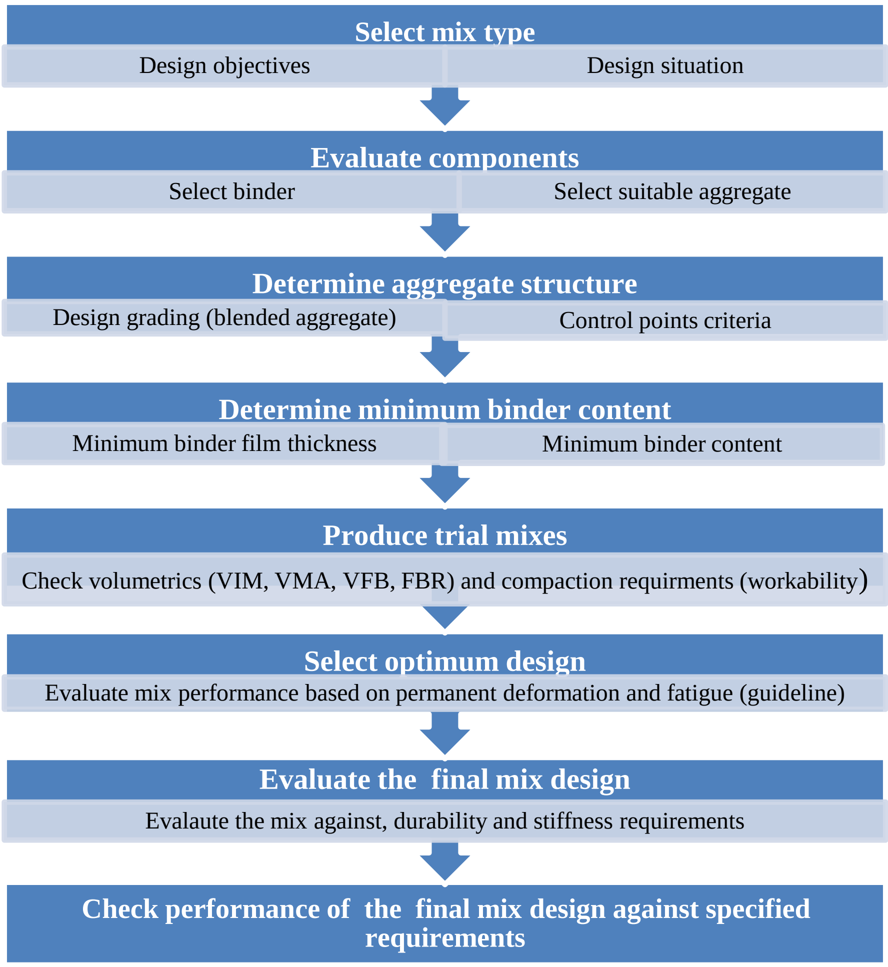


Figure 8: Level II and Level III design process

The basic steps involved in the Level II and Level III mix designs are given below:

(1) Select optimum mix. The selection of optimum design at these levels involves the same sample preparation and determination of volumetrics as described for Level I except that only the Superpave gyratory compactor (AASHTO T 312) test procedure is used. VMA is checked at the design binder content to verify that the requirements in Table 21 are met.

Compaction and VFB requirements for Level II and Level III as presented in Table 27 and Table 28 are different.

Table 27: Laboratory compaction requirements for Levels II & III

| Design Level | Design traffic [E80s] | $N_{design}$ |
|---|---|---|
| **II** | **3 to 30 million** | 100 |
| **III** | **> 30 million** | 125 |

Table 28: Percent VFB (Heavy to very heavy traffic)

| Design Level | Design traffic [E80s] | Minimum | Maximum |
|---|---|---|---|
| **II** | **3 to 30 million** | 65 | 75 |
| **III** | > **30 million** | 65 | 75 |

(2) The workability test is conducted on short-term aged gyratory compacted specimens of dimensions 150 mm diameter by 115 ± 2 mm high as per AASHTO T 312 testing procedures. Evaluate the mix as follows: The voids of the specimen after 45 gyrations should not exceed the design voids by more than three percent.

**Note 5.15:** These workability criteria serve as a guide only and clearly should be considered in conjunction with a number of factors such as shape and surface texture of available aggregate and mix type (i.e. stone or sand skeleton).

(3) Evaluate durability of the mix by using the Modified Lottman test procedures (ASTM D4867M), and check results against the criteria set in Table 25.

(4) Evaluate stiffness (expressed as dynamic modulus) of the mix at in-place voids in accordance with the procedures contained in AASHTO T 378.[o]

**Note 5.16:** At Level II design, dynamic modulus test is conducted at frequency sweeps of 0.1, 0.5, 1, 5, 10, and 25 Hz at one test temperature of 20°C. At Level III design, a full factorial test of dynamic modulus is conducted at the five frequencies above and at five temperatures (-5, 5, 20, 40 and 55°C).

(5) Select the optimum mix design based on performance

- ***Permanent deformation (rutting)*** – To evaluate the resistance of the mix to permanent deformation specimens are tested as per ASP4 of Sabita Manual 39. Tests are carried out at three binder contents. These binder contents are:
  - the optimum binder content at 4% voids obtained from the volumetric design procedures
  - optimum – 0.5%, and
  - optimum + 0.5%.

The Hamburg Wheel-Tracking Test (HWTT) is carried out in accordance with AASHTO: T 324[p] on two specimens for each binder content. For design purposes the test specimens shall be cylindrical laboratory prepared specimens using the gyratory compactor (Sabita Protocol ASP4). The specimen thickness shall be at least twice the nominal maximum aggregate size.

**Note 5.17**: The Hamburg Wheel Tracking Test (HWTT) indicates susceptibility to premature failing of asphalt mixtures due to weak aggregate structure, inadequate binder stiffness, moisture damage, and inadequate adhesion between aggregate and binder. HWTT results are influenced by aggregate quality, binder stiffness, duration of short-term ageing, binder source, anti-stripping treatments and compaction temperature.

[o] Although the AASHTO T 378 test standard is specific on the use of the Asphalt Mix Performance Tester (AMPT), testing systems other than the AMPT device are available.

[p] While AASHTO: T 324 requires laboratory specimen conditioning according the AASHTO: R 30, specimens should be prepared in accordance with Sabita Testing Protocol ASP 4.

The test outputs include post-compaction consolidation, usually assessed at 1 000 wheel passes, creep slope, stripping slope, and stripping inflection point, as illustrated in Figure 9.

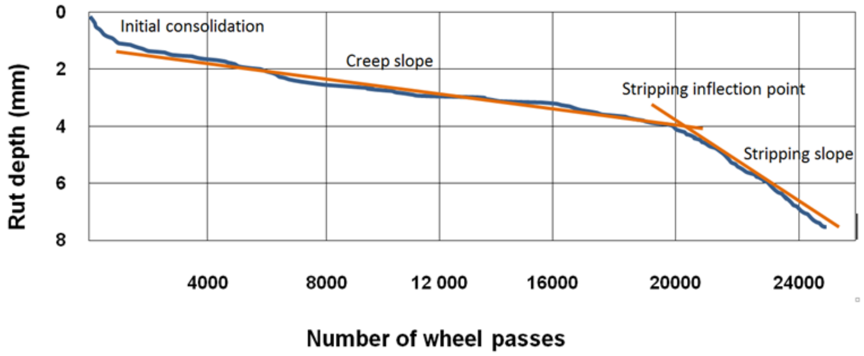


Figure 9: Definition of the Hamburg Wheel Tracking Test results

The compliance criteria are related to the PG maximum pavement design temperature zones (see section 3) and design traffic, as follows:

- For asphalt to be placed in the 58 maximum pavement design temperature zone and for design traffic up to 10 million E80 the minimum number of passes to comply with the prescribed maximum rut depth is 16 000. In the event where the design traffic is in excess of 10 million E80, the minimum number of passes to comply with the prescribed maximum rut depth is 20 000.
- For asphalt mixes to be placed in the 64 and 70 maximum pavement design temperature zones, the minimum number of passes to comply with the prescribed maximum rut depth is 20 000, regardless of traffic class;

These compliance criteria are set out in Table 29.

Work done in the USA suggests that the stripping inflection point occurring at less than 10,000 passes is an indication of moisture susceptibility.

Recording and assessing mix performance only on the basis of the final data after the prescribed number of wheel passes might be misleading. Factors such as the post compaction consolidation, the creep slope (no of wheel passes per mm rut depth) as well as the stripping slope should be considered when assessing data.

It is suggested that, in cases where the rut depth developed after the prescribed minimum number of passes exceeds 6 mm, further investigation be carried out , e.g. MMLS testing (see note 5.20).

**Note 5.18**: HWTT data validation/confirmation is to be performed on plant trial material and included for the determined working mix design selected for the project. Care should be taken to note the age condition of the mixture tested to allow informed analysis/comparison of results to be done between laboratory design conditions and plant manufactured conditions.

Table 29: Criteria for permanent deformation

<table>
<tr><th>Design Traffic [million E80]</th><th>Passes to 6 mm rut [Temperature zone]</th><th>SIP [min]</th><th>Mix Design Level</th></tr>
<tr><td rowspan="2">>3 - 10</td><td>16000 [58]</td><td rowspan="3">10 000</td><td rowspan="2">II</td></tr>
<tr><td>20 000 [64, 70]</td></tr>
<tr><td>> 10</td><td>20 000 [58, 64, 70]</td><td>II, III</td></tr>
</table>

- ***Fatigue Life –*** This property of the mix is assessed using the design binder content obtained from permanent deformation evaluation. Fatigue is evaluated in a four-point beam fatigue testing procedures as described in AASHTO T 321.
    1. Prepare slabs from compacted mix and cut the beams (380 mm long by 63 mm wide by 50 mm high) to conduct the fatigue test. A minimum of 9 specimens are prepared and tested at the design voids and design binder content for Level II design and a minimum of 27 specimens for Level III design, based on three repeat tests per test condition.
    2. For Level II design, conduct the fatigue test at one test temperature of 10°C and a loading frequency of 10 Hz at three strain levels to generate a fatigue curve for the mix (three repeats).
    3. For Level III design, conduct the fatigue test at three test temperatures of 5, 10 and 20°C at 10 Hz at three strain levels to generate fatigue curves for the mix (three repeats).
    4. The selected strain levels will depend largely on the type of binder in the mix. For conventional binders initial peak-to-peak strain levels within the range of 250 to 750 με may be appropriate; mixes with highly modified binders may require initial strain levels as high as 2 000 με. In accordance with AASHTO 321 strain levels should be such that at least 10 000 load cycles are applied. Accordingly it is considered that a practical upper limit of duration for testing a specimen should be two days (48 hours).
    5. Fatigue failure is defined as the load cycle ($n$) at which the product of the specimen stiffness ($S$) and the loading cycles, i.e. ($S \times n$) is a maximum with respect to $n$. (See Figure 10)
    6. Mix-specific fatigue models to estimate fatigue performance of an asphalt pavement layer can be derived in conjunction with the dynamic modulus of the mix as part of the pavement design process.

(6) Conduct air permeability test on the design mix in accordance with Sabita Manual 39: Protocol ASP10 and check results against the criteria presented in

(7) Table 26.
(8) Mix acceptance – The final mix design will be accepted when it meets all requirements /criteria presented in the pavement design process. If any of the requirements /criteria cannot be met, then consider adjustments to be made in aggregate or binder type, and aggregate grading in the mix design procedures.

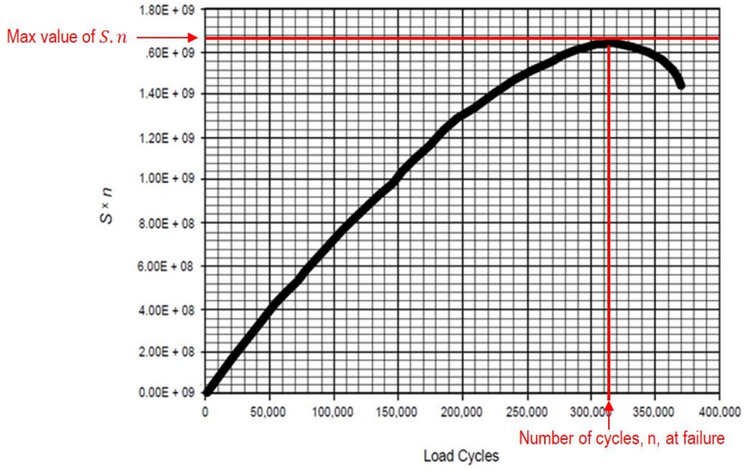


**Figure 10: S x n versus Load Cycles**

**Note 5.19:** All specimens compacted for the three mix design levels must be short-term aged (the procedure adopted in this manual requires 4 hours of short term ageing in a forced-draft oven at the compaction temperature, regardless of the aggregate absorption).

**Note 5.20:** Although not a specific requirement, the use of the MMLS3 as per testing protocol SANS 3001-PD1:2014 is recommended should additional investigation and site validation testing be indicated. (See note 5.18). Table 30 lists test properties testing conditions, and the number of compacted specimens required to conduct laboratory test for Level II and Level III designs.

**Table 30: Summary of performance-related tests**

| Property | Test conditions | No. of specimens | Test method |
|---|---|---|---|
| **Workability** | Superpave gyratory compactor, air voids after specified number of gyrations | 3 | AASHTO T 312 |
| **Durability** | Modified Lottman test conditions | 6 | ASTM D 4867M |
| **Stiffness/ (dynamic modulus)** | AMPT dynamic modulus at temperatures of -5, 5, 20, 40, 55°C; loading frequencies of 25, 10, 5, 1, 0.5, 0.1 Hz | 5 | AASHTO T 378 |
| **Permanent deformation** | HWTT at relevant number of passes. | 2[q] | AASHTO T 324 |
| **Fatigue** | Four-point beam fatigue test at maximum of three strain levels and three temperatures. | 9[r] | AASHTO T 321 |

## 5.7 Design of special mixes

A number of useful guidelines and production methodologies with recommendations and criteria are available for the following special mixes to supplement this design manual.

### 5.7.1 Cold mixes

[q] Number of specimens per binder content
[r] This is the number of specimens required for Level II design. For Level III design it will be necessary to prepare 27 specimens to enable the designer to repeat a test at a specific temperature and / or strain level (three repeats at each test condition).

Reference documents: TG2 Interim guideline 2002.

### 5.7.2 Porous asphalt

Additional mix design process and procedures are presented in SABITA Manual 17: *Porous asphalt mixes - design and use*.

### 5.7.3 Mixes for light traffic in residential areas

Reference document: Sabita Manual 27: *Guideline for thin layer hot mix asphalt wearing courses of residential streets*.

### 5.7.4 Warm mix asphalt

Reference document: Sabita Manual 32: *Best practice guide for warm mix asphalt*.

### 5.7.5 EME asphalt

Additional mix design process and procedures are presented in SABITA Manual 33: *Design procedure for high modulus asphalt.*

### 5.7.6 Mixes with reclaimed asphalt

Reference document: TRH 21*: 2016: Use of reclaimed asphalt in the production of asphalt*
The following limitations on the percentages of reclaimed asphalt in a mix are recommended:

- SMA – 0%
- Porous asphalt – 20%
- General asphalt – 50%

### 5.7.7 Stone mastic asphalt (SMA)

A guideline on the principles of the design of this type of mix is presented in Appendix B.

# 6. Link with asphalt pavement design

## 6.1 South Africa pavement design method

A new pavement design method referred to as South African Pavement Design method (SAPDM) and based on mechanistic-empirical relationships is due for implementation in due course. Some of the key factors that lead to this development are:

- Need for the utilisation of unconventional materials (new generation materials, recycled, cementitious stabilised, industrial wastes, marginal materials, etc.);
- Effects of the environment and traffic loading on pavement materials in order to relate structural response of the pavement to performance realistically;
- Use of fundamental asphalt material properties to predict resilient response and damage behaviour of the pavement, and
- Calibration of performance / damage models for the prediction of permanent deformation (rutting) and fatigue cracking of asphalt in the pavement system.

## 6.2 Asphalt pavement layer considerations

The asphalt layers (wearing course or base course) should be considered as elements of a pavement structure system in which substrate support will influence the magnitude of induced stresses and strains in the asphalt layer(s). This, in turn, will determine pavement response parameters in terms of elastic deflection basin parameters such as maximum deflection and radii of curvature.

Provided that they are well supported by substrates of adequate stiffness, asphalt layers of thickness > 35 mm can be considered as structural layers. The thicker asphalt layers reduce stresses and strains within the pavement and render such asphalt layers more resistant to fatigue cracking than thinner layers. Typically, this will result in lower maximum deflections and larger radii of curvature.

Additionally, stiffer asphalt base layers, e.g. EME, will deflect less under traffic loading and, in view of both their inherent stiffness and superior load spreading capacity, can be expected to experience relatively low stresses and strains, with associated benefits in both fatigue life and rutting.

The following models for asphalt materials are included in the revised SAPDM [53]:

- Resilient response;
- Damage;
  - Fatigue;
  - Plastic strain;

## 6.3 Resilient response of asphalt

The SAPDM requires the determination of dynamic modulus for resilient response characterisation of the asphalt materials regardless of the analysis level.

The following important models will be used in the SAPDM for asphalt materials:

- Binder ageing model;
- Dynamic modulus models;

The three design levels adopted in the SAPDM will require distinct methods for establishing a dynamic modulus for asphalt related to the risk profile adopted.

### 6.3.1 Basic level

At this level values used are based on the results of previously tested mixes. Model coefficients would be pre-loaded in the system

### 6.3.2 Intermediate level

Predictive empirical models for resilient response, based on component material-specific properties would be adopted. Models currently under consideration are the Witczak and Hirsh dynamic modulus predictive models.

### 6.6.3 Advanced level

At this level, the resilient modulus and performance characteristics would be determined by laboratory testing of project-specific mixes as detailed below.

### 6.6.4 Predicting dynamic modulus from laboratory data

Evaluation of dynamic modulus test results from laboratory involves generating master curves. The master curve of asphalt allows comparisons to be made over extended ranges of test temperatures and load frequencies.

Step-by-step procedures for the development of master curves for South Africa asphalt mixes are reported by Anochie-Boateng et al. (2010). The shape of the master curve is defined by a sigmoidal model shown in Eq. 6.6.

$$\log\left|E^*\right| = \delta + \frac{\alpha}{1+e^{\beta+\gamma \log f_r}} \qquad \text{(Eq. 6.6)}$$

where:

$|E^*|$= dynamic modulus [MPa]

$f_r$ .= reduced frequency [Hz]

$\delta$ = minimum value of $|E^*|$

$\delta + \alpha$ = maximum value of $|E^*|$

$\beta, \gamma$ = parameters describing the shape of the sigmoidal function

The reduced frequency (Eq. 6.7) is defined as the actual loading frequency multiplied by the time-temperature shift factor, *a* (*T*).

$$f_r = a(T) \times f \qquad \text{(Eq. 6.7)}$$

where:

$f$ = frequency [Hz]

$a\ (T)$ = shift factor as a function of temperature [ºC]

$T$ = temperature [ºC]

Optimization procedures in Microsoft Excel solver can be used to simultaneously determine the optimum values for the fitting parameters for Eq. 6.6 and Eq. 6.7, by maximizing the coefficient of determination ($R^2$) of the fit.

An example of the fitted curve parameters for the master curve is shown in Figure 11. The figure shows that the master curve is obtained by shifting the dynamic modulus results of different temperatures to form a smooth function with the results at the chosen reference temperature (in this case, 20ºC).

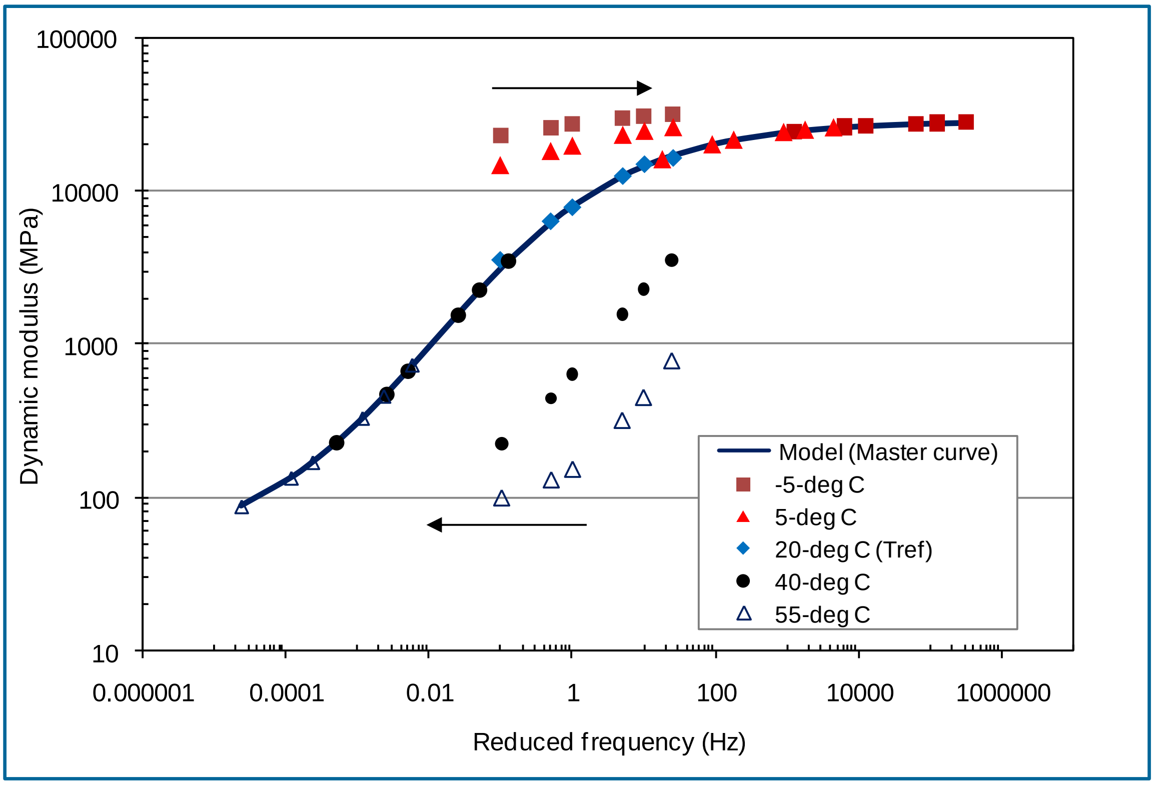


Figure 11 : Typical master curve for dynamic modulus (Anochie-Boateng et al. 2011)

Where the designer needs to examine the dynamic modulus at a temperature different to the one adopted as a reference temperature, use is made of the shift factors as shown in Figure 12.

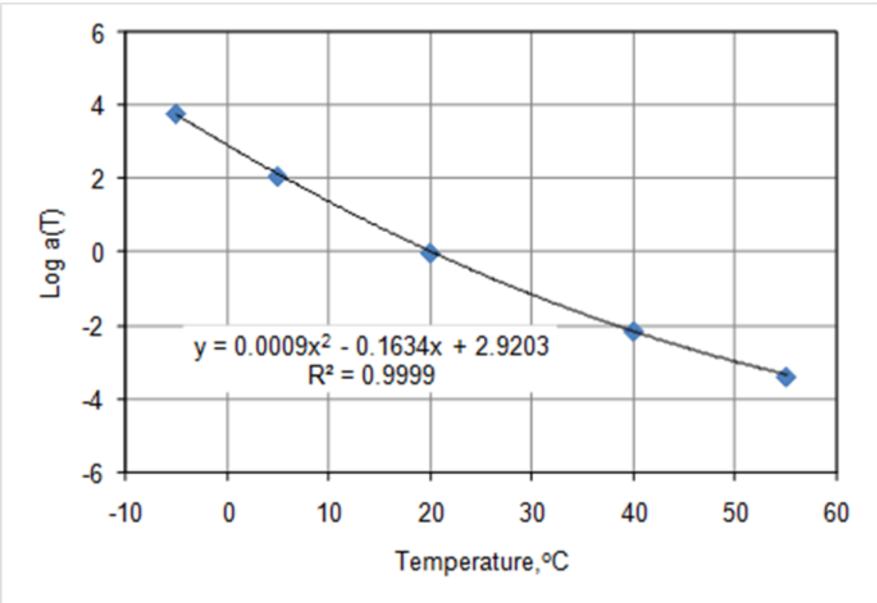


Figure 12: Temperature shift factors for master curve (reference temperature – 20°C)

## 6.4 Damage modelling

One of the major fundamental changes to the flexible pavement design method is the departure from the critical layer, end-of-life approach adopted previously and the move towards recursive cumulative damage simulation. Notwithstanding this approach, the criteria given in this manual for mix design related to adequate resistance to both fatigue damage and permanent deformation are essentially related to damage after a number of stress repetitions.

The assessment of the performance characteristics for a given pavement design situation, at a particular level of investigation, is considered to fall beyond the scope of this document.

## 6.5 Long life pavement

The purpose of mix design for asphalt in long life pavements is to determine the proportion of asphalt binder and aggregate that will give long lasting performance of the pavement system. The concept of long life pavement uses a thick asphalt layer over a firm foundation design with two asphalt layers (surfacing/wearing course, and base course); each one tailored to resist specific stresses.

- The surfacing course mix should be designed to provide adequate functional (see Chapter 2) and structural performance;
- The base course is the main structural layer. The mix should be designed to absorb load stresses and to limit strain responses in the pavement by distributing the applied loads over a wider area. In so doing, the base course will act against mechanisms that cause asphalt confined rutting;
- The base course asphalt should be designed to be a fatigue-resistant and durable layer. The following approaches can be used to resist fatigue cracking in the base course.
    - If the layer depth is sufficiently large or the layer stiffness sufficiently high, the tensile strain at the bottom of the base layer is insignificant (concept of endurance limit);
    - Additional flexibility can be imparted to the asphalt base layer through increasing the binder content and/or using a modified binder e.g. an elastomer type;
    - Combinations of the two approaches also work.

**Note 6.7:** Pavement considerations that need to be taken into account during the mix design stage of long life pavements are essentially the same as those for conventional pavements.

# 7. Quality Management Procedures

## 7.1 General

It is recommended practice that, after the successful design of a new mix in a laboratory, a trial mix is produced to assess workability and comparison of *in situ* properties of the mix with those of the laboratory produced specimens. Upon successful completion of the trial section, plant production and mix paving commences as per contractual requirements.

A complete quality management process is required from the asphalt mix design stage, to manufacturing and to actual paving to ensure that the design, manufacture and the actual paving of asphalt mix takes place in a prescribed manner which would guarantee that the specification requirements are met.

This chapter describes two key elements of a quality management process – quality control and quality assurance – required to ensure that the specified requirements of the asphalt mix are readily achieved.

## 7.2 Definitions

### 7.2.1 Quality control

Quality control of asphalt mix refers to those measures and procedures during manufacture, paving and compaction that are in place to ensure that the approved project mix materialises on site and that the contract specifications will be met. Typically, the processes involve monitoring the quality of component materials (binder, aggregate and filler), plant controls for mix proportions and field control during paving and compaction. Quality control is monitored in terms of pre-defined properties such as aggregate properties, binder content and grading.

A critical element of this process is the regular monitoring of aggregate stockpiles to ensure that materials being mixed are representative of those used in the project mix design. If at any time it is evident that this condition is not being met, a new design based on materials currently available should be submitted.

### 7.2.2 Quality assurance

This aspect of quality management covers measures and procedures to assess the quality of an asphalt mix placed in terms of compliance with the specified parameters such as mix characteristics and/or performance attributes.

## 7.3 Levels of mix design

Three asphalt mix design levels are considered in this manual (Chapter 5). These are:

- Volumetric design for low to medium volume roads (Level I). A mix design is usually tendered for each contract and client or consultant approval is obtained for the mix design.
- Performance-related mix designs (Level II and Level III). This approach is new and the design is dependent on relatively lengthy performance related laboratory testing. It would not be practical to repeat such designs on a contractual basis and it is proposed that individual suppliers would have a number of performance-related mixes certified for specific applications and performance expectations. Such certification would be valid for a period of two years if there were no significant changes to the raw materials used in such a certified mix. Where a performance-related mix is not certified, i.e., a purpose-designed mix, a 'certification-type' testing procedure precedes the quality control process, so the same quality control approach is still followed.

The approach to quality control during asphalt manufacturing and paving depends on the asphalt mix design approach. In this chapter, quality control procedures for both approaches are discussed.

The processes for the different levels of mix design are presented schematically in Table 31, along with parameters needed to be controlled at each major step. The parameters form the bases of the quality control processes to be implemented at each step.

Table 31: Mix design levels

| Level I | Levels II, III |
|---|---|
| **Contract based mix design**<br>• Aggregate properties, grading, binder content, VIM, MVD ,VMA, VFB, BD, ITS, dynamic creep, durability and permeability<br>↓ | **Certified mixes (or purpose-designed mixes)**<br>• Aggregate properties, grading, dynamic modulus, fatigue, permanent deformation, workability, durability, binder content, MVD and VIM<br>↓ |
| **Plant mix and trial section**<br>• Binder content, grading, VIM, MVD, VMA, VFB, compaction density<br>↓ | **Trial section**<br>• Grading, binder content and VIM/field density<br>↓ |
| **Site**<br>• Binder content, grading, VIM, compaction density, layer thickness<br>• Frequency of sampling and acceptance limits are defined in the relevant specifications | **Site**<br>• Grading, binder content and VIM/ field density<br>• Paving – QC: compaction, temperature control, limiting segregation, layer thickness |

## 7.4 Mix design level I

Typically, the process consists of a laboratory mix design, plant trial, construction of trial paving section and site paving.

### 7.4.1 Laboratory design

The mix design involves selection and proportioning of materials (binder, aggregate and filler) such that the desired mix properties are obtained.

The design procedures are described in Chapter 5. The final optimum mix is defined in terms of parameters including binder content, voids (VIM), voids in the mineral aggregate (VMA), voids filled with binder (VFB), indirect tensile strength (ITS), dynamic creep, permeability and modified Lottman.

Table 32 gives typical specification requirements for each parameter.

Table 32: Level I design: materials, mix characteristics and specifications at the design stage

| Property | | Specification/design/report values |
|---|---|---|
| Binder | Binder grading (SANS) | Compliance with specification grading as per relevant standard (Proof of specs on compliance usually given by binder supplier) |
| | Binder testing confirmation | Softening Point, penetration and viscosity (Confirmation of specification certificate) |
| Aggregate / Filler | BD / AD | Report Values |
| | Voids in Compacted Filler | Compliance with the requirements given in Table 8 and Table 14. |
| | Density in Toluene | |
| | ACV | |
| | 10% FACT | |
| | Magnesium Sulphate soundness | |
| | Methylene blue Adsorption / Test | |
| | FI | |
| | PSV | |
| | Fractured faces | |
| | Water absorption | |
| | Clay lumps and friable Particles | |
| | Sand equivalent | |
| | Grading | Compliance with project mix design grading |
| Binder content | | Optimum design value evaluated |
| Design voids @ optimum binder content | | |
| VMA | | Compliance with the requirements given in Table 26 |
| VFB | | |
| ITS | | |
| Dynamic creep | | |
| Permeability | | |
| Modified Lottman (TSR) | | |
| VMD | | |
| $BD_{MIX}$ | | Report Only |
| | | |

### 7.4.2 Plant mix

The optimum laboratory mix is manufactured at a plant, and the mix parameters are determined. The parameters include grading, binder content, binder absorption, voids, voids in the mineral aggregate (VMA), voids filled with binder (VFB), indirect tensile strength (ITS), dynamic creep, permeability and modified Lottman. This serves as a verification of the laboratory design.

To ensure that production conditions can be representatively replicated the quantity of asphalt to be mixed at each design point shall not be less than 20 tons provided that the associated production run time is not less than 10 minutes.

### 7.4.3 Trial section

Once the plant mix has been approved, a trial section is constructed to assess field performance of the mix. The trial section aims at assessment of mix constructability, test properties of field samples and to establish the required compaction effort. The asphalt mix parameters are established, and tolerances for acceptance control are set. Table 33 shows the material properties and mix characteristics to be assessed, as well the permissible deviations.

**Table 33: Level I design: Permissible deviation from the design at the trial section**

| Property | | Permissible deviation from design |
|---|---|---|
| Binder content | | The binder content should be within the limits specified.<br>*Alternatively*<br>± 0.3% for continuous and semi-gap graded mixes<br>± 0.4% for gap graded and bitumen rubber mixes |
| Grading<br><br>(percentage passing sieve size) | Sieve size (mm) | |
| | 28 | ±5.0% |
| | 20 | ±5.0% |
| | 14 | ±5.0% |
| | 10 | ±5.0% |
| | 7,1 | ±5.0% |
| | 5 | ±4.0% |
| | 2 | ±4.0% |
| | 1 | ±4.0% |
| | 0,6 | ±4.0% |
| | 0,3 | ± 3.0% |
| | 0,15 | ± 2.0% |
| | 0,075 | ± 1.0% |
| VIM | | ± 1.5% |
| VMA | | Compliance with specification requirement as given in Table 26 |
| VFB | | |
| ITS | | |
| Dynamic creep | | |

| | |
|---|---|
| Permeability<br>Modified Lottman (TSR) | |
| Compaction Density | The density shall be within the limits specified<br>*Alternatively*<br>Minimum: (97% - % design voids ) of MVD<br>Maximum: 96% of MVD |

The quantity of a trial mix depends on a number of factors including the capacity of the plant and contractual requirement. COTO recommends that 300 m³ to 600 m³ of trial section be constructed.

### 7.4.4 Field/site: Quality control

After successful evaluation of the trial section, the approved asphalt mix becomes the project mix. During paving, certain mix characteristics are monitored to assess their compliance with the project mix specifications. The monitored mix characteristics include binder content, grading and voids/density. Testing Frequency and acceptance limits are shown in Table 34. Layer thickness and levels are also monitored.

**Table 34: Level I design: Permissible deviations from design / contract specifications at the paving stage as well as the testing frequency**

| Property | | Permissible deviation | Testing frequency |
|---|---|---|---|
| Binder content | | The binder content shall be within the limits specified in the applicable statistical judgment scheme<br>*Alternatively*<br>± 0.3% for continuous and semi-gap graded mixes,<br>± 0.4% for gap graded and bitumen rubber mixes | 6 per lot * |
| Grading (percentage passing sieve size) | Sieve size (mm) | | |
| | 28 | ±5.0% | 6 per lot* |
| | 20 | ±5.0% | |
| | 14 | ±5.0% | |
| | 10 | ±5.0% | |
| | 7,1 | ±5.0% | |
| | 5 | ±4.0% | |
| | 2 | ±4.0% | |
| | 1 | ±4.0% | |
| | 0,6 | ±4.0% | |
| | 0,3 | ± 3.0% | |
| | 0,15 | ± 2.0% | |

| | | | |
|---|---|---|---|
| | 0,075 | ± 1.0%** | |
| VIM | | ± 1.5% | 2 per lot* |
| Density/voids in mix[3] | | The density shall be within the limits specified in the applicable statistical judgment scheme[3]<br><br>*Alternatively*<br><br>Minimum: (97% - % design voids) of MVD<br><br>Maximum: 96% of MVD | 4 per lot* |
| Layer thickness | | The layer thickness shall be within the limits specified in the applicable statistical judgment scheme | One day's work |

* A construction lot is a section that is constructed at the same time, of the same materials, and using the same method. It is considered to be the same for testing purposes. A lot is generally about a day's work or an element of a structure.

** When statistical methods are applied, the permissible deviation for 0,075 mm fraction is ± 2.0%.

## 7.5 Level II and Level III design

The performance-related approach is closely associated with the concept of certified mixes. The proposed quality control procedures proposed for a certified mix is based on the assumption that if the constituent material (binder and aggregate/filler) properties and mix characteristics (binder content and grading) do not change, then the performance-related parameters of the mix should not differ significantly from the certified properties.

### 7.5.1 Mix certification

The asphalt mix performance-related parameters that will be certified are:

- Dynamic modulus (value at field voids);
- Fatigue (value at design voids);
- Permanent deformation (value at field voids);
- Workability value, and
- Durability (TSR value field voids).

The performance-related parameters are evaluated after simulation of short-term ageing and they should comply with the minimum requirements provided by the client / contract. The certification will be associated with specific material properties (aggregate/filler and binder) and certain mix characteristics as defined in Table 35.

Table 35: Material properties and mix characteristics to be certified

| Component | Material property/Mix characteristic | Specification/certified/report values |
|---|---|---|
| Aggregate/filler | BD / AD | Report Values |
| | ACV | Compliance with specification requirement As given in section 4 |
| | 10% FACT | |
| | Magnesium Sulphate soundness | |
| | Methylene blue adsorption | |
| | FI | |
| | PSV | |
| | Fractured faces | |
| | Water absorption | |
| | Clay lumps and friable Particles | |
| | Sand equivalent | |
| | Bailey parameters | |
| | Grading | |
| Binder | Grade of binder | (Proof of specification compliance usually given by binder supplier) |
| Mix | Binder content | Report Value |
| | Design voids @ $N_{design}$ | Report Value |

### 7.5.2 Trial section

The evaluation of the performance-related parameters (dynamic modulus, fatigue, permanent deformation, workability, and durability) will not be repeated. The assumption is that mix characteristics including grading, binder content, density and voids should be strictly controlled to ensure that the performance-related parameters are maintained. Therefore, the grading, binder content, density and voids are the trial section mix characteristics that will be assessed. These properties should not deviate significantly from the certified values.

Table 36 shows permissible deviation of mix properties.

**Table 36: Level II and Level III design – permissible deviation from the certified values at the trial section**

| Property | | Permissible deviation from certified values |
|---|---|---|
| Binder Grade/Type | | Compliance with specification required |
| Binder content | | The binder content shall be within the limits specified<br>*Alternatively*<br>± 0.3% for continuous and semi-gap graded mixes,<br>± 0.4% for gap graded and bitumen rubber mixes |
| Grading<br>(percentage passing sieve size) | Sieve size (mm) | |
| | 28 | ±5.0% |
| | 20 | ±5.0% |
| | 14 | ±5.0% |
| | 10 | ±5.0% |
| | 7,1 | ±5.0% |
| | 5 | ±4.0% |
| | 2 | ±4.0% |
| | 1 | ±4.0% |
| | 0,6 | ±4.0% |
| | 0,3 | ± 3.0% |
| | 0,15 | ± 2.0% |
| | 0,075 | ± 1.0% * |
| Design voids @ $N_{design}$<br>(compacted loose mix) | | Design value ± 1.5% |
| Density of the paved mix[s] | | The density shall be within the limits specified<br>*Alternatively*<br>Minimum: (97% - % design voids ) of MVD<br>Maximum: 96% of MVD |

* When statistical methods are applied, the permissible deviation for 0,075 mm fraction is ± 2.0%.

### 7.5.3 Site/field: Quality control

During the asphalt paving, the mix characteristics including grading, binder content, density and voids shall be monitored to ensure that the performance-related properties are met. Similar to the trial section, the field mix characteristics should not differ significantly from the certified values. The permissible deviation from the certified mix and the required test frequencies are shown in Table 37.

[s] For the compaction density requirements of EME, see Sabita Manual 33

**Table 37: Level II and Level III design: Permissible deviations from certified values at the paving stage as well as testing frequency**

| Property | | Permissible deviation from certified/contractual values | Testing frequency |
|---|---|---|---|
| Binder Grade/Type | | Compliance with specification required | Ongoing |
| Binder content | | The binder content shall be within the limits specified in the applicable statistical judgment scheme.<br>*Alternatively*<br>± 0.3% for continuous and semi-gap graded mixes<br>± 0.4% for gap graded and bitumen rubber mixes | 6 per lot * |
| Grading<br>(percentage passing sieve size) | Sieve size (mm) | | |
| | 28 | ±5.0% | 6 per lot * |
| | 20 | ±5.0% | |
| | 14 | ±5.0% | |
| | 10 | ±5.0% | |
| | 7,1 | ±5.0% | |
| | 5 | ±4.0% | |
| | 2 | ±4.0% | |
| | 1 | ±4.0% | |
| | 0,6 | ±4.0% | |
| | 0,3 | ± 3.0% | |
| | 0,15 | ± 2.0% | |
| | 0,075 | ± 1.0% ** | |
| Density of the paved mix *** | | The density shall be within the limits specified<br>*Alternatively*<br>Minimum: (97% - % design voids) of MVD<br>Maximum: 96% of MVD | 4 per lot * |
| Layer thickness | | The layer thickness shall be within the limits specified in the applicable statistical judgment scheme | One day's work |

* A construction lot is a section that is constructed at the same time, of the same materials, and using the same method. It is considered to be the same for testing purposes. A lot is generally about a day's work or an element of a structure.

** When statistical methods are applied, the permissible deviation for 0,075 mm fraction is ± 2.0%.

*** For the compaction density requirements of EME, see Sabita Manual 33

## 7.6 Test methods

Table 38 presents the list of test methods for evaluation of material properties, mix characteristics and performance-related parameters.

Table 38: Test methods

| Category | Property | Test method |
|---|---|---|
| Aggregate/filler | Bulk Density in Toluene | BS 812 |
| | Voids in Compacted Filler | BS 812 |
| | Fines aggregate crushing value (10% FACT) | SANS 3001-AG10 |
| | Aggregate crushing value (ACV) | SANS 3001-AG10 |
| | Ethylene glycol durability index | SANS 3001-AG14 |
| | Durability mill index values | SANS 3001-AG16 |
| | Aggregate impact value (AIV) | BS 812: Part 3 |
| | Flakiness index test | SANS 3001-AG4 |
| | Polished stone value Test (PSV) | BS 812-114 |
| | Coarse aggregate bulk density, apparent density and water absorption | SANS 3001-AG20 |
| | Fine aggregate bulk density, apparent density and water absorption | SANS 3001-AG21 |
| | Magnesium soundness | SANS 3001-5839 |
| | Sand equivalent | SANS 3001-AG5 |
| | Fractured faces | SANS 3001 AG4 /TMH1/ASTM D 5821 |
| | Methylene blue adsorption / test | SANS 6243 |
| | Clay lumps and friable Particles | ASTM C142 |
| | Grading | SANS 3001-AG1 |
| Mix characteristics | Binder content | SANS 3001-AS20 |
| | Binder absorption | SANS 3001 AS11 |
| | Grading | SANS 3001-AS20 |
| | VIM | SANS 3001 AS10 |
| Mix performance parameters | Dynamic modulus | CSIR SANRAL/ AASHTO T 378 |
| | Fatigue | AASHTO T 321 |
| | Permanent deformation | AASHTO T 324 |
| | Workability | AASHTO T312 |
| | Durability | ASTM D4867M |

| | | |
|---|---|---|
| | ITS | ASTM 6931 |
| | Dynamic creep modulus | CSIR RMT-004 |
| | Permeability | Sabita Manual 39: ASP 5 |

## 7.7 Asphalt paving and construction factors affecting quality control

To ensure that mix and structural design objectives are fulfilled, it is essential that the asphalt layers are evenly paved *to the correct thickness* and compacted to an adequate initial density before opening to traffic. Every effort should therefore be made on site to employ procedures and processes that will safeguard the attainment of these goals. The subject of sound construction practice is comprehensively covered in Sabita Manual 5: *Guidelines for the manufacture and construction of hot mix asphalt*. However, some aspects are covered here in view of their critical function in assuring that overall objectives are attained.

### 7.7.1 Paving

Level control is critical and it is recommended that some form of automatic level control be used on both sides of the paver, while noting that the level control measure may be different on each side, necessitating the use of different equipment.

The uncompacted mat behind the screed must be paved thicker than the final required thickness as compaction reduces the paved, "loose" thickness. The degree of reduction in thickness differs for various asphalt mixes, and typical examples are given in Table 39. These are guideline figures and actual reduction figures should be determined on site.

Table 39: Reduction of loose paved thickness

| Material | Reduction |
|---|---|
| **Asphalt bases** | 25 to 30% |
| **Continuous graded wearing course** | 17 to 20% |
| **Open-graded, UTFC** | 8 to 10% |

### 7.7.2 Compaction

Compaction is the most important factor required to ensure that the performance-related properties of asphalt mixes are achieved. Asphalt compaction is affected by a number of factors including:

- Material properties (aggregate, binder and mix properties);
- Environmental variables (layer thickness and weather conditions e.g. rain, temperature and wind);
- Site conditions, and
- Type of compaction equipment.

Best practices required to ensure that adequate compaction is achieved include:

- Equipment selection (pavers and rollers);
- Sequence of compaction equipment ;
- Rolling patterns and speed;
- Correct roller operation, and
- Timing, from batching to paving
- In the case of WMA, care should be taken to ensure that the mat is not over-compacted.

### 7.7.3 Temperature

During asphalt paving, temperature control is important. Inappropriate compaction temperature could result in problems such as difficulty in achieving the required density, water permeability etc. Ageing of the binder is also affected by the mix temperature, which ultimately affects the performance-related parameters. Therefore, temperature measurements should be done for each load of mix arriving on site.

### 7.7.4 Segregation

It is important to ensure that segregation of the mix does not occur. Segregation results in variability of the composition of the layer, i.e. binder content and aggregate particle size distribution. The finer fraction of the asphalt mix will yield binder contents higher than the mean content while a coarser portion results in a lower binder content. Segregation may also result in variation of density and voids, as well as the overall performance of the mix.

Segregation may be exaggerated especially during loading and paving of large aggregate mixes (See Sabita Manual 5)

## 7.8 Functional mix acceptability

In addition to satisfactory structural performance of paved asphalt, the paved sections should yield acceptable functional performance. The functional performances indicators include:

- Surface texture for adequate skid resistance and limited noise generation (especially in urban areas);
- Riding quality;
- Appearance, and
- Noise generation.

Detailed discussion on how to ensure that these aims are achieved, fall outside of the scope of this manual. However, users of this manual are encouraged to consult relevant documents/guidelines, which cover these aspects in detail.

# 8. References and bibliography


1. CSIR South African asphalt mix design manual JK Anochie-Boateng, J O'Connell, BMJA Verhaeghe, E Denneman, JW Maina, January 2014
2. AASHTO PP 60. *Provisional standard practice for preparation of cylindrical performance test specimens using the Superpave gyratory compactor*, AASHTO, Washington DC. USA.
3. AASHTO T 19M/T 19-09. *Standard method of test for bulk density ("Unit weight") and voids in aggregate*. AASHTO, Washington DC. USA.
4. AASHTO T 312-12. *Standard method of test for preparing and determining the density of hot-mix asphalt (HMA) specimens by means of the Superpave gyratory compactor*, AASHTO, USA.
5. AASHTO PP 60-13. Standard Practice for Preparation of Cylindrical Performance Test Specimens Using the Superpave Gyratory Compactor (SGC). AASHTO, Washington DC. USA.
6. AASHTO T321-14. *Standard method of test for determining the fatigue life of compacted hot-mix asphalt subjected to repeated flexural bending*, AASHTO, Washington DC. USA.
7. AASHTO T 378. *Standard method of test for determining the dynamic modulus and flow number for asphalt mixtures using the asphalt mixture performance tester (AMPT)*, AASHTO, USA.
8. Anochie-Boateng, J., and Maina, J. (2012). *Permanent deformation testing for a new South African mechanistic pavement design method.* Construction and Building Materials, 26(1), 541-546.
9. Anochie-Boateng, J., Denneman, E., O'Connell, J., and Ventura, D. 2010*. Hot-mix asphalt testing for the South African pavement design method*. Proceedings of 29th Southern Africa transportation conference, Pretoria.
10. Anochie-Boateng, J., O'Connell, J., Denneman, E and B. Verhaeghe. 2011. *Resilient response characterization of hot-mix asphalt mixes for a new South African pavement design method*. The 10th Conference on Asphalt Pavements for Southern Africa, Champagne Sports Resort, KZN, Sept 2011.
11. ASTM D 4867M. *Standard test method for effect of moisture on asphalt concrete paving mixtures*. ASTM International, West Conshohocken, PA, USA. www.astm.org
12. ASTM D3398 -00. *Standard test method for index of aggregate particle shape and texture*. ASTM International, West Conshohocken, PA, USA. www.astm.org.
13. ASTM D6925-09. *Standard Test Method for Preparation and Determination of the Relative Density of Hot Mix Asphalt* (*HMA*) *Specimens by Means of the Superpave Gyratory Compactor.* ASTM International, West Conshohocken, PA, USA. www.astm.org
14. ASTM D7313 - 07a. *Standard Test Method for Determining Fracture Energy of Asphalt-Aggregate Mixtures Using the Disk-Shaped Compact Tension Geometry*, AASHTO, Washington DC. USA.
15. Aurilio, V., Pine, W.J., and Lum, P., 2005. *The Bailey method. Achieving volumetrics and HMA compactability*.
16. Barksdale D., 1971. *Compressive stress pulse times in flexible pavements for use in dynamic analysis*. Highway Research Record, 345, Highways Research Board, Washington, pp32-44.
17. Brown, S.F., 1973. *Determination of Young's modulus for bituminous materials in pavement design*. Highway Research Record, 431, Highways Research Board, Washington, pp38-49.
18. Christensen, D.W., Pellinen, T.K., and Bonaquist, R.F. 2003. *Hirsch Model for Estimating the Modulus of Asphalt Concrete*. Journal of the Association of Asphalt Paving Technologists, Volume 72, Lexington.
19. COTO. 1998. *Standard Specifications for Road and Bridge Work for State Road Authorities*. South African Institution of Civil Engineers. South Africa.
20. Denneman, 2007. *The application of locally developed pavement temperature prediction algorithms in performance grade (PG) binder selection*. Proceedings of the 26th Southern African Transport Conference, Pretoria.
21. Manual L 1/2002. *Asphalt Manual*. Gauteng Provincial Government: Department of Public Transport, Roads and Works. Directorate Design. Pretoria. South Africa.
22. National Cooperative Highway Research Program (NCHRP) Report 614. 2008. *Refining the simple performance tester for use in routine practice*, Washington, D.C, USA.

23. National Cooperative Highway Research Program (NCHRP). 2004. *Guide for mechanistic-empirical design of new and rehabilitated pavement structures*. Final report of NCHRP 1-37A. Washington D.C., USA.
24. Prowell, B.D. July 2007. *Warm Mix Asphalt*. The International Technology Scanning Program Summary Report. Federal Highway Authority, USA
25. Sabita Manual 17. January 2011. *The Design and Use of Porous Asphalt Mixes*. Sabita. Cape Town South Africa. www.sabita.co.za
26. Sabita Manual 18. January 1996. *Appropriate Standards for the Use of Sand Asphalt*. Sabita. Cape Town South Africa. www.sabita.co.za
27. Sabita Manual 19. March 2009. *Guidelines for the Design, Manufacture and Construction of Bitumen Rubber Asphalt Wearing Courses*. Sabita. Cape Town South Africa.
28. Sabita Manual 2. August 2007. Bituminous Binders for Road Construction and Maintenance. Sabita. Cape Town South Africa. www.sabita.co.za
29. Sabita Manual 25. July 2005. *User Guide for the Design of Hot Mix Asphalt*. Sabita. Cape Town South Africa. www.sabita.co.za
30. Sabita Manual 27. May 2008. *Guide line for thin layer hot mix asphalt wearing courses on residential streets*. Sabita. Cape Town South Africa. www.sabita.co.za
31. Sabita Manual 32. September 2011. *Best practice guideline for warm mix asphalt*. Sabita. Cape Town South Africa. www.sabita.co.za
32. Sabita Manual 33. January 2013. *Interim Design Procedure for High Modulus Asphalt*. Sabita. Cape Town South Africa. www.sabita.co.za
33. Sabita Manual 5. March 2008. *Guidelines for the manufacture and construction of hot mix asphalt*. Sabita. Cape Town South Africa. www.sabita.co.za
34. SANRAL South African Pavement Engineering Manual, January 2013
35. SANS 3001-AG10. Civil engineering test methods Part AG10: *ACV (aggregate crushing value) and 10% FACT (fines aggregate crushing test) values of coarse aggregates*, Pretoria. www.sabs.co.za
36. SANS 3001-AG11. Civil engineering test methods Part AG11: *Polished stone value*, Pretoria. www.sabs.co.za
37. SANS 3001-AG12. Civil engineering test methods Part AG12: *Soundness of aggregates (magnesium sulphate method)*, Pretoria. www.sabs.co.za
38. SANS 3001-AG144. Civil engineering test methods Part AG14: *Determination of the ethylene glycol durability index for rock*, Pretoria. www.sabs.co.za
39. SANS 3001-AG20. Civil engineering test methods Part AG20: *Determination of the bulk density, apparent density and water absorption of aggregate particles retained on the 5 mm sieve for road construction materials*, Pretoria. www.sabs.co.za
40. SANS 3001-AG21. Civil engineering test methods Part AG21: *Determination of the bulk density, apparent density and water absorption of aggregate particles passing the 5 mm sieve for road construction materials*, Pretoria. www.sabs.co.za
41. SANS 3001-AG4. Civil engineering test methods Part AG4: *Determination of the flakiness index of coarse aggregate*. Pretoria. www.sabs.co.za
42. SANS 3001-AG5. Civil engineering test methods Part AG5: *Sand equivalent value of fine aggregates*, Pretoria. www.sabs.co.za
43. SANS 3001-AS1. Civil engineering test methods Part AS1: *Making of asphalt briquettes for Marshall tests and other specialized tests*, Pretoria. www.sabs.co.za
44. SANS 3001-AS10. Civil engineering test methods Part AS10: *Determination of bulk density and void content of compacted asphalt*, Pretoria. www.sabs.co.za
45. SANS 3001-AS11. Civil engineering test methods Part AS11: *Determination of the maximum void-less density of asphalt mixes and the quantity of binder absorbed by the aggregate*, Pretoria. www.sabs.co.za
46. SANS 5839. *Soundness of aggregates (magnesium sulphate method)*. Pretoria. www.sabs.co.za
47. SANS 6243. *Deleterious clay content of the fines in aggregate (methylene blue adsorption indicator test)*. Pretoria. www.sabs.co.za

48. South African National Road Agency Limited (SANRAL). 2007. *Revision of South African pavement design method*, Report PB/2006/B-4: A Design Input System for Road-Building Material, Pretoria.
49. Taute, A., Verhaeghe, B., & Visser, A., 2001. Interim guidelines for the design of hot-mix asphalt in South Africa. Pretoria.
50. Technical Guideline: TG 1. 2007. *The use of modified bituminous binders in road construction*, Second Edition, Asphalt Academy, Pretoria, South Africa.
51. Vavrik, W.R., Huber, G., Pine, W.J., Carpenter, S.H., and Bailey, R., 2002. *Bailey method for gradation selection in hot-mix asphalt mixture design*. Transportation Research E-Circular, Number E-C044, October 2002, Washington D.C., USA.
52. Viljoen, A. W., 2001, *Estimating Asphalt temperatures from air temperatures and basic sky parameters*. Internal report, Transportek, CSIR, Pretoria.
53. Kannemeyer et al, Revision of South African pavement design method, Conference on Asphalt Pavements for Southern Africa, Sun City, August 2015
54. van de Ven MFC, Smit ADF and Lorio R, *Stone Mastic Asphalt Mix Design Based on a Binary System* Annual Transport Convention 1998, Pretoria.
55. National Asphalt, Mix design procedure for SMA, January 2010
56. Australian Asphalt Pavement Association, Implementation Guide No.4, *Stone Mastic Asphalt Design & Application Guide* 2000.
57. Liljedahl B, *What is a Stone Mastic Asphalt?* European Asphalt Magazine, EAPA, 1995.
58. Romero P, VanFrank K and Nielson J *Analysis of the Hamburg Wheel Tracking Device to Predict Behavior of Asphalt Mixtures at Different Test Temperatures* Utah Department of Transportation Salt Lake City, October, 2008
59. Aschenbrener, Terrel and Zamora *Comparison of the Hamburg Wheel Tracking Device and the environmental Conditioning System to Pavements of Known Stripping Performance*, Colorado Department of Transportation, Denver, January 1994.
60. Yildirim Y and Kennedy TW *Correlation of Field Performance to Hamburg Wheel Tracking Device Results,* Texas Department of Transportation Austin, October 2001
61. Rahman F and Hossain M *Review and Analysis of Hamburg Wheel Tracking Device Test Data*, Kansas Department of Transportation Topeka, February 2014

# Appendix A– Overview of the Bailey Method for Determining Aggregate Proportions

While it has been noted in section *4.5 Grading requirements* that some parameters of this method are based on aggregates encountered in the USA, its application in South Africa should be approached with some caution and should preferably be used by experienced designers only. Nevertheless, the method will provide valuable guidance in determining the proportioning of asphalt mixes for a wide range of applications and instil an enhanced understanding of aggregate packing configurations that are not possible by assessing particle size distributions only.

## A.1 Aggregate grading

The Bailey method may be used to evaluate three types of asphalt mixes (fine-graded, coarse-graded and SMA).

## A.2 Definitions

- Coarse aggregates – particles that when placed in a unit volume creates voids.
- Fine aggregates – particles that can fill the voids created by the coarse aggregate in the mix
- Half sieve – the closest sieve to one half the NMPS.
- Primary control sieve (PCS) – the sieve that controls the designation between coarse and fine aggregates. PCS is the closest sieve to 22 percent of the nominal maximum particle size (Eq. A.1).
- Secondary control sieve (SCS) – the closest sieve to 22 percent of the primary control sieve size.
- Tertiary control sieve (TCS) –the closest sieve to 22 percent of the secondary control sieve.

$$PCS = 0.22 \times NMPS \qquad \text{(Eq. A.1)}$$

The 22 percent used to determine the Bailey control sieves is determined from the estimation of void size created by the four aggregate shape combinations.

## A.3 Unit weight of aggregates

Unit weight is the traditional terminology used to describe the property determined in the Bailey method, which is weight per unit volume (mass per unit volume or density). Table 40 shows unit weights and test methods used in the Bailey concepts. Table 41 presents recommended chosen unit weights of mix types, whereas the characteristics of the various mix types are presented in Table 43.

Table 40: Bailey unit weights and test methods

| Unit weight | Characteristics | Test method | Criteria |
|---|---|---|---|
| **Loose unit weight (LUW)** | • No compactive effort<br>• Start of particle-to-particle contact<br>• Determine LUW (kg/m³)<br>• Determine volume of voids | AASHTO T19 | $V_{LUW}$[1] : 43% – 49%<br>$V_{LUW} = 100 \times \left[\frac{BD_A - LUW}{BD_A}\right]$ |
| **Rodded unit weight (RUW)** | • Requires compactive effort<br>  ○ Three layers<br>  ○ Rodded 25 times each<br>• Increased particle-to-particle contact<br>• Determine RUW (kg/m³)<br>• Determine volume of voids | AASHTO T19 | $V_{RUW}$[2]: 37% – 43%<br>$V_{RUW} = 100 \times \left[\frac{BD_A - RUW}{BD_A}\right]$ |
| **Chosen unit weight (CUW) (Table 4-5)** | • Value that the designer selects based on the desired interlock of coarse aggregate<br>• The designer must decide the desired mix type; fine-graded, coarse-graded or a stone mastic mix<br>• After the mix type is selected, the percent chosen unit weight can be selected | N/A | **Table 4-5** |

[1]: $V_{LUW}$ = Loose unit weight voids; $BD_A$ = Bulk density of aggregate;

[2] $V_{RUW}$ = Rodded unit weight voids

Table 41: Recommended chosen unit weights

| Mix type | Unit weight | CUW % |
|---|---|---|
| **Fine-graded** | CA LUW | 60 - 80 |
| **Coarse-graded** | CA LUW | 95 to 105 |
| **SMA** | CA RUW | 110 to 125 |

CA = Coarse aggregate.

**Note**: The term “unit weight” is used in the reference material for the Bailey method, although the value is actually density since the units are kilograms per cubic meter. The common term of unit weight is used throughout the text to comply with the convention.

## A.4 Loose and rodded unit weight voids

The loose unit weight voids is derived from the loose unit weight, and the bulk relative density of the coarse aggregate as presented in Eq. A.2. Similarly, the rodded unit weight voids is derived from the rodded unit weight, and the bulk relative density of the coarse aggregate as presented in Eq. A.3. Typical ranges of voids are presented in Table 42.

$$V_{LUW} = 100 \times \left[\frac{RDA - LUW}{RDA}\right] \quad \text{(Eq. A.2)}$$

$$V_{RUW} = 100 \times \left[\frac{RDA - RUW}{RDA}\right] \quad \text{(Eq. A.3)}$$

where:

$V_{LUW}$ = Loose unit weight voids
$V_{RUW}$ = Rodded unit weight voids
$LUW$ = Loose unit weight
$RUW$ = Rodded unit weight

*RDA* = Bulk relative density of aggregate

Table 42: Recommended unit weight voids

| Aggregate fraction | LUW voids range | RUW voids range |
|---|---|---|
| **Fine-aggregates** | 35% - 43% | 28% - 36% |
| **Coarse-aggregates** | 43% - 49% | 37% - 43% |

Table 43: Characteristics of mix types

| Mix type | Characteristics |
|---|---|
| **Fine-graded** | • Coarse aggregate volume < LUW<br>• Little to no particle-to-particle contact of coarse aggregate<br>• Fine fraction carries most of the load |
| **Coarse-graded** | • Coarse aggregate volume ≈ LUW (95 – 105)<br>• Some particle-to-particle contact of coarse aggregate<br>• Coarse and fine fractions carry load |
| **SMA** | • Coarse aggregate volume ≫ RUW<br>• Coarse fractions carries load<br>• Remaining voids filled with mastic |

## A.5 Aggregate packing analysis

The design and analysis of an aggregate blend is built on three important ratios:

1. Coarse aggregate (CA) ratio – describes grading of the coarse aggregate; how the coarse aggregate particles pack together and, consequently, how these particles compact the fine aggregate portion of the aggregate blend that fills the voids created by the coarse aggregate.
2. $FA_c$ ratio– describes the grading of the coarse portion of the fine aggregate; how the coarse portion of the fine aggregate packs together and, consequently, how these particles compact the material that fills the voids it creates.
3. $FA_f$ ratio– describes the grading of the fine portion of the fine aggregate; how the fine portion of the fine aggregate packs together. It also influences the voids that will remain in the overall fine aggregate portion of the blend because it represents the particles that fill the smallest voids created.

$$CA\ ratio = \frac{\text{Percentage passing half sieve} - \text{Percentage passing PCS}}{100 - \text{Percentage passing half sieve}} \quad \text{(Eq. A.4)}$$

$$FA_c\ ratio = \frac{\text{Percentage passing SCS}}{\text{Percentage passing PCS}} \quad \text{(Eq. A.5)}$$

$$FA_f\ ratio = \frac{\text{Percentage passing TCS}}{\text{Percentage passing SCS}} \quad \text{(Eq. A.6)}$$

Table 44, Table 45 Table 46 show the control sieves and recommended aggregate ratios for fine-graded, coarse graded and SMA mixes, respectively.

Table 44: Control sieves for fine-graded mixes

| NMPS (mm) | Original PCS (New NMPS) | New Half sieve | New PCS | New SCS | New TCS |
|---|---|---|---|---|---|
| **37,5** | 10 | 5 | 2 | 0.6 | 0.15 |
| **28** | 7,1 | 2 | 1 | 0.3 | 0.075 |
| **20** | 5 | 2 | 1 | 0.3 | 0.075 |
| **14** | 2 | 1 | 0.6 | 0.15 | --[1] |
| **10** | 2 | 1 | 0.6 | 0.15 | --[1] |
| **7,1** | 1 | 0.6 | 0.3 | 0.075 | --[1] |
| **5** | 1 | 0.6 | 0.3 | 0.075 | --[1] |

[1]Sieve sizes too small for values to be determined.

Table 45: Control sieves for coarse-graded mixes

| (NMPS, mm) | Half sieve | PCS | SCS | TCS |
|---|---|---|---|---|
| **37,5** | 20 | 10 | 2 | 0.6 |
| **28** | 14 | 7.1 | 1 | 0.3 |
| **20** | 10 | 5 | 1 | 0.3 |
| **14** | 7,1 | 2 | 0.6 | 0.15 |
| **10** | 5 | 2 | 0.6 | 0.15 |
| **7,1** | 2 | 1 | 0.3 | 0.75 |
| **5** | 2 | 1 | 0.3 | 0 075 |

Table 46: Control sieves for SMA mixes

| (NMPS, mm) | Half sieve | PCS | SCS | TCS |
|---|---|---|---|---|
| **20** | 10 | 5 | 1 | 0.3 |
| **14** | 7,1 | 2 | 0.6 | 0.15 |
| **10** | 5 | 2 | 0.6 | 0.15 |
| **7,1** | 2 | 1 | 0.3 | 0.075 |
| **5** | 2 | 1 | 0.3 | 0.075 |

**Note**: PCS, SCS and TCS constitute the control sieves when using the Bailey concepts, similar to the conventional way of aggregate blending in which the NMPS, 2 mm, and 0,075 mm sizes for instance, are critical sieves for control (target) points.

## A.6 Effects of aggregate ratios on VMA

Table 47 and Table 48 present the recommended aggregate ratios for different NMPS. The effect of aggregate ratios on the VMA is dependent on whether the aggregate blend is considered fine or coarse by Bailey definition.

Table 47: Recommended ranges for aggregate ratios in fine and coarse mixes[1]

<table>
<tr><th rowspan="2">NMPS (mm)</th><th rowspan="2">CA (coarse-graded)</th><th rowspan="2">New CA ratio</th><th colspan="2">Coarse and fine -graded</th></tr>
<tr><th>$FA_c$</th><th>$FA_f$</th></tr>
<tr><td>37,5</td><td>0.80–0.95</td><td rowspan="7">0.60-1.00</td><td rowspan="7">0.35–0.50</td><td rowspan="7">0.35–0.50</td></tr>
<tr><td>28</td><td>0.70-0.85</td></tr>
<tr><td>20</td><td>0.60-0.75</td></tr>
<tr><td>14</td><td>0.50-0.65</td></tr>
<tr><td>10</td><td>0.40-0.55</td></tr>
<tr><td>7,1</td><td>0.35-0.50</td></tr>
<tr><td>5</td><td>0.30-0.45</td></tr>
</table>

[1] These ranges provide a starting point where no prior experience exists for a given set of aggregates. If the designer has acceptable existing designs, they should be evaluated to determine a narrower range to target for future designs.

Table 48: Recommended ranges for aggregate ratios in SMA mixes

| NMPS (mm) | CA | $FA_c$ | $FA_f$ |
|---|---|---|---|
| 20 | 0.35-0.50 | 0.60-0.85 | 0.65-0.90 |
| 14 | 0.25-0.40 | 0.60-0.85 | 0.65-0.9 |
| 10 | 0.15-0.30 | 0.60-0.85 | 0.65-0.9 |

**Note**: These ratios have been reviewed in the light of the SANS sieve sizes which came into effect in 2013. As a consequence the $FA_f$ ranges have been adjusted.

Table 49 shows the general effect on the VMA based on changes in the aggregate ratios. Also, the change in value of the Bailey parameters resulting in a 1% change in VMA is shown in Table 50.

Table 49: Effect of VMA - changes in aggregate ratios

| | Fine-graded | Coarse-graded | SMA |
|---|---|---|---|
| **CA** | increase | increase | increase |
| **$FA_c$** | decrease | decrease | decrease |
| **$FA_f$** | decrease | decrease | decrease |

Table 50: Change in value of Bailey parameters to produce 1% change in VMA

| | Fine-graded | Coarse-graded |
|---|---|---|
| **CA** | 0.35 | 0.20 |
| **$FA_c$** | 0.05 | 0.05 |
| **$FA_f$** | 0.05 | 0.05 |

**Note:** Bailey ratios are calculated based on aggregate grading. The effect of change in grading on VMA is similar to the effect of change in the Bailey aggregate ratios on VMA.

**Note:** Changes in the new ratios for fine-graded mixes create similar results in regards to the VMA.

## A.7 Procedure to blend aggregates

The designer needs the following information:

- Grading and the bulk density of aggregate fractions (SANS 3001-AG1, SANS 3001-AG20/ AG21), and,
- Loose and rodded unit weights (AASHTO T-19).

The designer should also decide on the following for the individual aggregate fractions:

- Chosen unit weight as a percentage of the loose unit weight;
- Desired percent passing 0,075 mm sieve;
- Blend by volume of coarse aggregates, and
- Blend by volume of fine aggregates.

Steps for blending aggregates using the Bailey method:

1. Conduct three laboratory tests on all aggregate fractions; (a) grading (b) BRD of aggregates, and (c) Unit weights - LUW, RUW.
2. For aggregates designed to obtain fine-graded mixes, select CUW (%) based on coarse aggregate LUW (Table A - 2)). On the other hand for aggregates designed to obtain SMA mixes the CUW is based on coarse aggregate RUW.
3. Determine the unit weight (LUW or RUW) contributed by each coarse aggregate according to the desired proportions (by volume) of coarse aggregate (*contribution = percent coarse aggregate x chosen unit weight*).
4. Determine the voids in each coarse aggregate according to its corresponding CUW and contribution by volume. Then sum the voids contributed by each coarse aggregate.
5. Determine the unit weight (LUW or RUW) contributed by each fine aggregate according to the desired proportions (by volume) of fine aggregate.
6. Determine the initial blend percentage by mass of each aggregate. Divide the mass of each aggregate fraction by the mass of the total aggregate blend.

7. Determine the amount of material passing 0,075 mm sieve contributed by each aggregate fraction.
8. Determine the amount of filler required, if any, to bring the percent passing the 0,075 mm sieve to the desired level.
9. Once the desired amount of material passing 0,075 mm sieve is achieved, adjust the final blend percentages (by mass) of fine aggregate fractions. In this step the blend percentage of coarse aggregate is not changed.
10. The final blending percentages (by mass) and aggregate ratios are determined and checked against Bailey requirements.

# Appendix B – Principles of the Design of Stone Mastic Asphalt

## B.1 Introduction

Stone Mastic Asphalt (SMA) is a premium asphalt wearing course possessing key functional, economic and technical advantages compared to conventional mixtures for surfacing. It is a durable material suited to high traffic volumes and, if properly designed yields an extended design life. Other, functional, advantages include:

- Superior skid resistance;
- Excellent ride quality;
- Low noise levels;
- Low tendency of back spray under wet conditions.

First introduced ca. 1970 by G Zichner in Germany, SMA is essentially a binary system comprising a self-supporting stone structure made up of particles larger than 2 mm, partially filled with binder-rich mastic. This configuration of mineral material classifies SMA as a stone skeleton mix type. The term self-supporting stone structure has no sense unless there is contact between the larger particles throughout the entire SMA layer and this contact is sufficiently stable to carry the traffic loading.

This stone skeleton is kept in place by the adhesion and cohesion of the mastic (i.e. the binder and the mineral aggregate finer than 2mm). It is of prime importance to compose the stone skeleton and the mastic in such a way as to retain the stone-to-stone contact intact, i.e. the stone skeleton should not be dilated by the mastic. The risk of undesirable dilation of the coarse particles will be minimised if the spaces in the stone skeleton are sufficiently large while the proportion of larger particles in the mastic component is kept low.

In an SMA the binder content is such as to form a voidless mastic in the mixture prior to compaction, which will ensure durability if the volume of the mastic and the coarse aggregate skeleton air voids are in proportion to each other. The air voids in the compacted mixture should be in the order of 3 %.

To prevent excessive draining of the binder during handling of the product the use of fibres or modification of the binder is often resorted to.

## B.2 Design approach

As there does not appear to be a universally accepted design method for SMA available, the purpose of this section is to set out the principles to be adopted in the design of this material, to ensure that key parameters are met. It is up to the designer to use the appropriate methods and procedures to ensure that these principles are achieved.

A design approach based on compliance with a grading envelope is discouraged as such an approach would not assure a mixture composition that meets the fundamental requirements of a stone skeleton, partially filled with mastic.

Consequently it is recommended that the design of SMA is tackled by either:

1. Application of the principles given in the Bailey method with a CUW of 110 – 125 %; or
2. A method based on a binary system (after Francken).

Option 1 can be followed by reference to Appendix A. A method based on a binary system is given below.

## B.3 Design method

The mix design steps to be taken into account using the binary system approach are:

1. design of the stone skeleton,
2. design of the mastic,
3. design of the mix.

Figure 13 below illustrates that the mix gradation is made up of the distinct gradings of the stone and mastic. The grading of coarse material will provide a stone skeleton and the grading for the fine material to form the mastic to partially fill the voids in the stone skeleton.

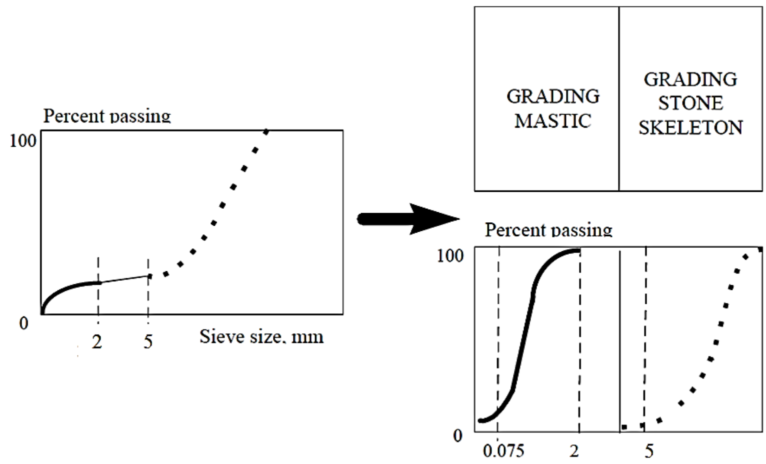


Figure 13: Mix gradation components

### B3.1 *Design of Stone Skeleton*

Based on the layer thickness to be used for the SMA surfacing a coarse aggregate (>2 mm)[t] grading must be chosen to justify a spatial approach based on a binary system of coarse aggregate and a mastic. For example for a 14 mm MPS (or 10 mm NMPS) the fractions between both the 0.600 mm – 2 mm and the 2 mm –5 mm sieves should be small. In other words, the grading of the aggregate should have a pronounced gap between 0,5 and 5 mm.

For the grading chosen, the voids in the coarse aggregate (VCA) are determined. Two methods are suggested:

1. Briquettes consisting only of coarse aggregate and low binder content (4%) are prepared and their volumetric properties determined. This includes the grading of the coarse aggregate before and after compaction to ensure that excessive degradation does not occur. If the grading of the mix after compaction changes significantly, replacement of the coarse aggregate may be necessary, or the change in grading should be anticipated on.
2. Determination of the volume of air in between the coarse aggregate particles when subjected to dry rodding in accordance with AASHTO T19.

### B3.2 *Design of the mastic*

The mastic plays a critical role in the performance of SMA, and also in the manufacturing and construction phase. The binder content is such that the filler-bitumen system is totally overfilled. Estimated on the fine aggregate exclusively, the binder content on the mastic of the SMA presented by Zichner was about 23 %.

The grading of the mastic can also be divided into two fractions, the fine aggregate (> 0,075 mm, <2 mm) and filler (< 0,075 mm). Research on fine aggregate/filler systems indicates that a minimum voids content is generally achieved when the ratio fine aggregate : filler is 4 : 1. This is demonstrated in Figure 14 below.

Since a separate fine aggregate skeleton is undesirable as it may adversely affect the stability of the stone skeleton, precautions should be taken to ensure that this situation does not arise. Consequently the mastic needs to be in a replacement state.

---

[t] While this criterion for distinguishing between coarse and fine aggregate is generally used, the designer may consider a larger sieve size where the maximum nominal particle size is 14 mm or greater.

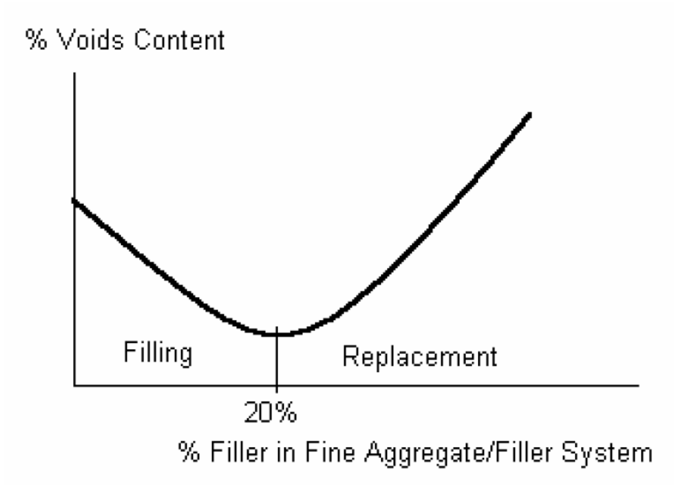


Figure 14: Influence of fine aggregate : filler ratio

Starting with 100% fine aggregate and gradually adding filler to it, the VMA of the fine aggregate/filler system can be determined, particularly the minimum VMA which will indicate a mode change from filling to replacement. This is necessary to achieve a replacement mode where there is no chance of developing a fine aggregate skeleton in between the voids of the coarse aggregate.

The mastic will be totally overfilled with bitumen and it is known from experience that sufficient bitumen will be available for coating the coarse aggregate.

### *B3.3 Design of the mix*

It is suggested that the volumetric properties of the mixes containing various proportions of coarse aggregates (> 2 mm), e.g. 65%, 70% and 75% be determined, while keeping the binder content and the fine aggregate/filler ration constant.

By changing the mastic content and, hence, the amount of free bitumen, the voids in the mix will vary. Figure 15 shows the relationship between voids and changing the coarse aggregate fraction while keeping the bitumen content constant.

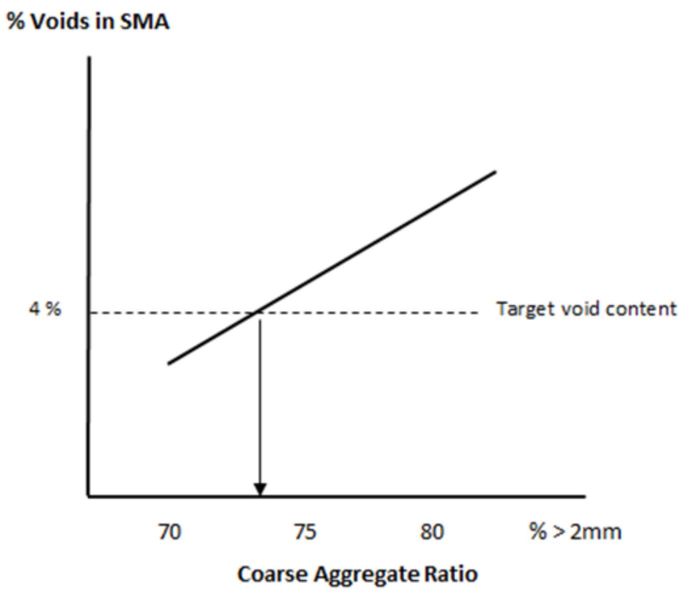


Figure 15: Relationship of voids and coarse aggregate ratio

The job mix proportions are based on the target voids content based on experience in the field. Figures ranging between 3 and 4,5 % have been proposed. This target voids content is also influenced by factors such as preventing dilation of the stone skeleton while retaining mix impermeability.

As mentioned before, a *fundamental requirement of an SMA is to ensure that the stone skeleton is not dilated by excessive mastic in the voids of the coarse aggregate*. For this purpose it should be ensured that the $VCA_{MIX}$ i.e. the volume in between the coarse aggregate particles, comprising filler, fine aggregate, air, binder, and (where used) fibre should be less than the VCA of the dry aggregate.

As illustrated in Figure 16 the coarse aggregate (> 2mm) should be at least 69%.

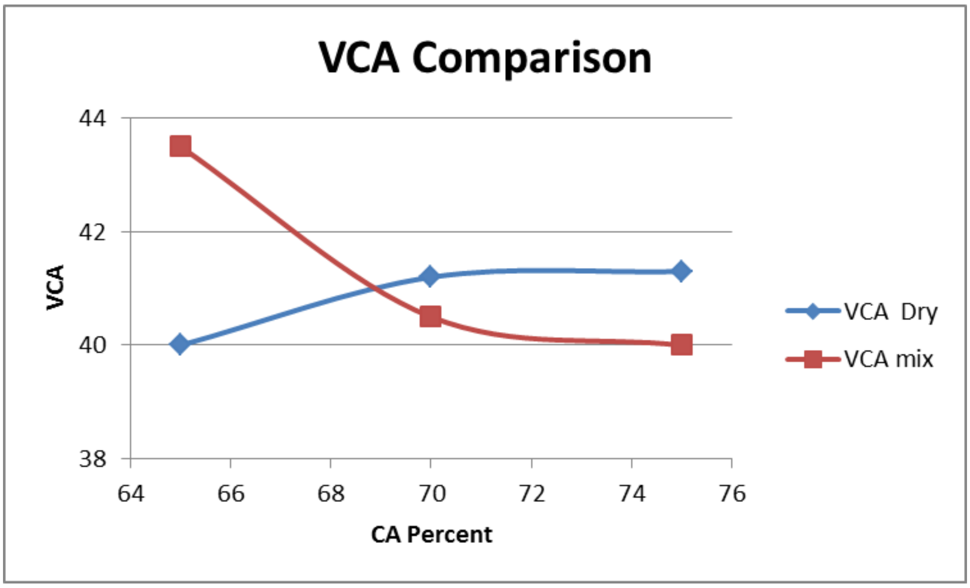


Figure 16: Comparison of $VCA_{dry}$ and $VCA_{mix}$

## B4 Additional tests

### *B4.1 Mastic run-off*

The overall viscosity of the mastic should be such that run-off during mixing, particularly transportation (especially over long distances) and paving is contained to within acceptable limits. Cellulose fibres (typically 0.3% to 0.5% m/m of the total mix) are widely used for this purpose. Alternatively, the use of a polymer modified binder may be considered.

A procedure similar to the one applied for open-graded asphalt the Schellenberg Drainage Test can be adopted to assess mastic run-off. This relatively simple test procedure entails 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 without shaking or vibrating it. The material retained in the receiver is weighed and the percentage weight loss is determined.

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.

Note that cellulose fibres can be damaged by high temperature and it is important that they do not come in contact with aggregates or drum *mix* gases at a temperature greater than 200°C. Such restrictions do not apply to mineral fibres such as rock wool and glass fibre.

### *B4.2 Moisture susceptibility*

As with other asphalt types the modified Lottman test (ASTM D4867 M) can be used to assess the moisture susceptibility of SMA. A minimum TSR of 70% should be achieved.