Asphalt mix design manual for South Africa

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Southern African Bitumen Association (Sabita)
Post Net Suite 56, Private Bag X21
Howard Place, 7450

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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.

ACKNOWLEDGEMENTS

LIST OF ABBREVIATIONS

Abbreviation	Definition
AASHTO	American Association of State Highway and Transportation Officials
AMPT	Asphalt Mixture Performance Tester
ASTM	American Society for Testing and Materials
COLTO	Committee of Local 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
HMA	Hot-Mix Asphalt
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
MPS	Maximum Particle Size
NMPS	Nominal Maximum Particle Size
PCS	Primary Control Sieve
QC / QA	Quality Control / Quality Assurance
RA	Reclaimed asphalt
RLPD	Repeated Load Permanent Deformation
RSST-CH	Repeated Simple Shear Test at Constant Height
RTFOT	Rolling Thin Film Oven Test
SAMDM	South African Mechanistic Design Method
SANAS	South African National Accreditation System
SANRAL	South African National Road Agency 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, except that procedures for mixes with reclaimed asphalts are given in TRH 21 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. Three 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 (Level I). 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. While the Bailey method, which has been used with success in South Africa, can be used to optimize aggregate grading, design criteria will not be set in this manual, as the criteria are based on the aggregates used for road construction in the USA. Nevertheless 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 link with pavement design. The traditional laboratory tests performed on a mix design could not be used to predict the performance of the mix in terms of elastic response, permanent deformation (rutting) and fatigue in a pavement structure. The revised *South African Pavement Design Method* (SAPDM) will allow this link between mix design and structural performance prediction. The material properties obtained from laboratory testing in a performance-related asphalt mix design can be used as input for the structural design methods.

An important component in the SAPDM will be the characterisation of the binder stiffness (and therefore changes in the resilient response of the mix) at different ages of the design life, using the dynamic shear rheometer (DSR). Ultimately, the DSR will be used in a performance grade binder selection process, which will replace the conventional penetration grade binder selection. Until the performance grade binder specification is fully implemented, it is proposed that DSR results are included in binder specification testing on a report only basis.

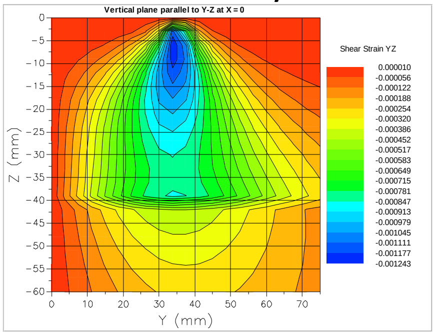
Figure 1-1 shows a hypothetical example of the mix selection process for medium to high volume roads. The performance requirements for the mix are determined based on mechanistic-empirical pavement analysis using the updated SAPDM software. Requirements are set for stiffness, permanent deformation and fatigue.

These requirements can then be included in the tender documentation, together with requirements for workability (an interim specification has been proposed whereby the voids at 25 gyrations should not exceed the design voids by more than 2%), and durability (tensile strength ratio in the modified Lottman test). Functional requirements such as skid resistance for the mix would also be specified for wearing course asphalt.

Mixes can be certified when they have gone through the process given in Figure 1-1 i.e. comprehensive performance related testing, where appropriate. The certification will be associated with specific materials (aggregate, filler and binder) and their properties and mix blend 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. The example in Figure 1-1 also shows how from three existing mix designs, a suitable mix would be selected. 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.

Pavement analysis



Structural requirements

Property	value
E* [GPa]	> 5
Fatigue [με to 10^6]	> 300
Perm. def. [ε_p]	< 2%

Tender specification

Property	value
E* [GPa]	> 5
Fatigue [με to 10^6]	> 300
Perm. def. [ε_p]	< 2%
Workability [voids]	< 6%
Durability [TSR]	> 80%

Mix selection

Property	Mix 1	Mix 2	Mix 3
E* [GPa]	14	6	3
Fatigue [με to 10^6]	220	370	280
Perm. def. [ε_p]	0.8 %	1.5 %	4.2 %
Workability [voids]	5.0	4.5	5.2
Durability [TSR]	90	85	75

Figure 1-1: Example of mix selection process

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 once the new asphalt design manual has been completed using Agrément South Africa or another vehicle. Agrément South Africa already provides fit-for-purpose certification for cold-mix asphalt and ultra-thin bituminous surfacing systems. Agrément typically uses the services of independent South African National Accreditation System (SANAS) accredited laboratories for the required testing.

1.6 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.

The properties determined using the performance-related tests in Chapter 5 form the input required for the asphalt pavement design models presented in Chapter 6.

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 main 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 mainly carried by the finer aggregate fraction, with the larger fractions providing bulk and replacing a proportion of the finer fraction. There is no meaningful contact between the individual larger aggregate particles. Examples include semi-gap graded asphalt, gap-graded asphalt, and medium / fine continuously graded asphalt.

2.1.2 Stone skeleton mixes

The spaces between the coarser aggregate fractions are filled by the finer aggregate fractions, but do not push the coarser aggregates apart. Contact between the coarser aggregate fractions is thus assured. This situation results in the loads on the layer being carried predominantly by a matrix (or skeleton) of the coarser aggregate fraction. Examples include coarse continuously graded asphalt, stone mastic asphalt, ultra-thin friction courses, and open graded asphalt (porous) asphalt.

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:

2.2.1.1 Heavy vehicles

For the purposes of mix design, traffic intensity / classes are evaluated using Table 1.

Table 1: Traffic classification

Design traffic [E80] [1]	Description
< 0.3 million	Low / Light
0.3 to 3 million	Medium
3 to 30 million	Heavy
≥ 30 million	Very heavy

[1] 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

2.2.1.1.1 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.

2.2.1.1.2 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.

2.2.1.1.3 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

1

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.

2.2.1.2 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.

2.2.1.3 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.

2.2.1.4 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 .

2.2.1.5 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.

2.2.2 Layer 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, 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, it is generally accepted that the maximum particle size (MPS) should be at most one third of the layer thickness to ensure compactability and to counter segregation during paving. As an example, for a 40 mm asphalt layer, the MPS should not exceed 14 mm or for a 30 mm layer the NMPS should not exceed 10 mm.

The recommended minimum layer thicknesses in relation to MPS as per Sabita Manual 5 are indicated in Table 2, and a typical nominal mix sizes for pavement applications are presented in Table 3. The *preferred layer thickness* in Table 2 should in particular be adopted where variation of layer thickness is likely to occur.

Table 2: Recommended minimum layer thickness

MPS [mm]	Minimum layer thickness (mm)	
	Absolute minimum	Preferred minimum
7,1	20	25
10	30	35
14	45	50
20	80	90
25	100	110

Table 3: Typical maximum particle sizes (MPS) for various applications in pavement

Mix type	Application	Traffic	MPS
Sand skeleton	Wearing course	Light / Low	7,1 mm; 10 mm
		Medium to heavy[1]	10 mm, 14 mm
		Very heavy	14 mm, 20 mm
	Base course[2]	All traffic conditions	10 mm, 14 mm, 20mm, 25 mm
Stone skeleton	Wearing course	All traffic conditions	10 mm, 14 mm
	Base course	All traffic conditions	14, 20 mm, 25 mm

[1]14 mm is generally preferred to 10 mm;

[2]: Better to use the largest practicable size that is economically justifiable.

2.2.3 Climatic considerations

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

2.2.3.1 Maximum temperature

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

2.2.3.2 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.

2.2.3.3 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.

2.2.3.4 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.

2.2.4 Other considerations

2.2.4.1 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.

Recommended mixes for improving skid resistance (friction) and noise reduction are provided in Table 7. Dust, spilled diesel, oil and excessive bitumen can significantly decrease skid resistance.

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 4 defines classes of texture and their characteristics.

Table 4: 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 2-1.

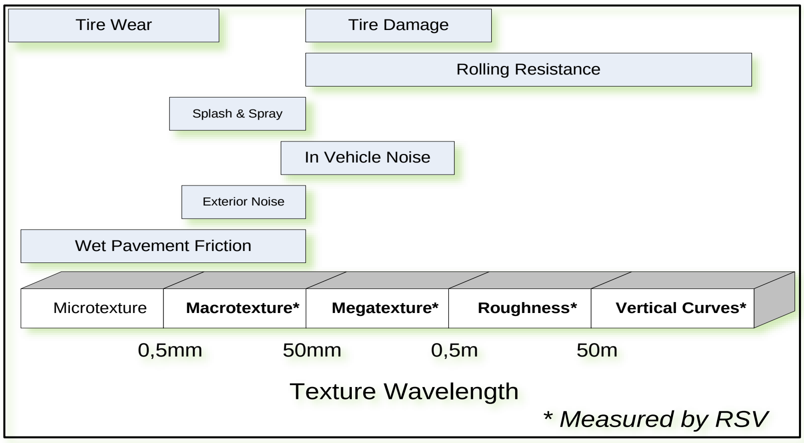


Figure 2-1 Functional requirements in relation to surface texture

2.2.4.2 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.

2.2.4.3 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 and bitumen materials commonly used for asphalt production in South Africa are given in **Table 5** and **Table 6** , respectively.

Table 5: 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				✓			✓		

Table 6: Types of bitumen used in asphalt and refineries[1]

Bitumen type	Grade /Class
Penetration grade bitumen	10/20
	15/25
	35/50
	50/70
	70/100
Modified bitumen	A-E1
	A-E2
	AP-1
	A-H1, A-H2
	A-R1

[1]: CALREF (Cape Town, Western Cape), ENREF (Durban, KwaZulu-Natal), NATREF (Sasolburg, Gauteng), SAPREF (Durban, KwaZulu-Natal).

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 in drums.

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.

2.3 Mix design consideration 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;
- Continuously graded asphalt can be manufactured with grading varying from very coarse to very fine, for a particular maximum aggregate size.

Table 7 shows some grading types for various applications. The ratings indicated range from poor (1) to excellent (4) and are based on generally held views of experienced practitioners. These ratings serve as a guide only and are not absolute nor restrictive.

Table 7: Mix types and typical performance ratings

Mix type	Binder type[1]	Typical application	Performance rating (1 = Poor; 4 = Excellent)				
			Rut resistance	Durability/ fatigue resistance	Skid resistance[2]	Impermeability to water	Noise reduction
Sand skeleton	Neat binder	Wearing course	2	2	2	3	2
	AR		3	4	2	3	2
	AE		3	3	2	3	2
	AP		4	3	2	3	2
	AH		3	3	2	3	2
	Rejuvenated		3	3	2	3	2

	(RA)						
Stone skeleton	Neat binder (Open-graded)		3	3	4	1[3]	4
	AE, AP (SMA)		4	4	3	3	4
	AE (Open-graded)		4	3	4	1[3]	4
	AR (Open graded)		4	4	4	1[3]	4
Sand skeleton	Neat binder		3	3		3	
	AE		4	4		3	
	AP		4	3		2	
Stone skeleton	10/20 pen (EME)		4	4		4	
	15/25 pen (EME)		4	4		4	
	AE		3	4		2	
	AP	Base layer	4	3	N/A	2	N/A

[1] The binder type refers to the generic descriptions only
[2].For semi-gap or gap graded mixes, the ratings for friction are based on layers with rolled-in-chips
[3] Impermeable support layer or membrane required

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, 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 manual, South Africa is in the process of translating from an empirical type bitumen specification to a performance grade 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 AsAc publication TG1 *The use of modified binders in road construction* will hold sway.

Performance grade specifications for binders focus on the evaluation of binder properties based on the traffic loading and environmental conditions (mainly temperature) which the binder will be subjected to in the field. 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 maps depicting the 7-day average maximum asphalt temperatures at 20 mm depth and the 1-day minimum asphalt temperatures at the surface are presented in Figure 3-1 and Figure 3-2.

Based on the maps in Figure 3-1 and Figure 3-2, South Africa can be divided into two performance graded (PG) binder zones based on the 7-day average maximum asphalt temperatures:

- **PG 58 Zone** which would include the Western Cape (except for the northern inland regions), Eastern Cape, most of KwaZulu-Natal, eastern half of the Free State, Gauteng, South Eastern part of Limpopo, and Mpumalanga (except for the eastern region bordering Mozambique).

- **PG 64 Zone** which covers the rest of the country, including the Northern Cape (except for the mountainous southern region), North West, the extreme northern coastal region of KwaZulu-Natal and rest of Limpopo.

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

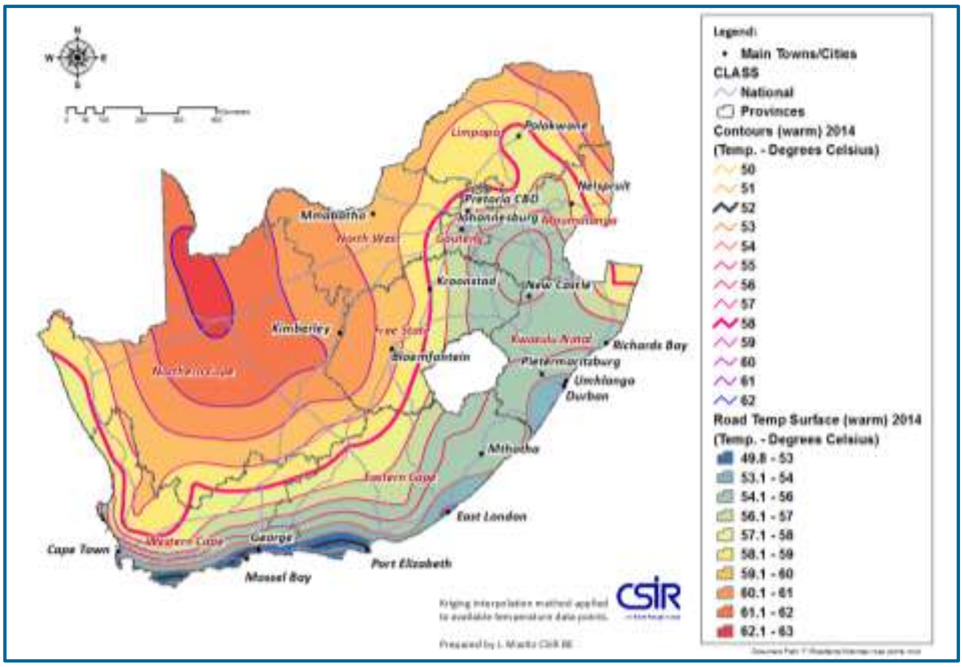


Figure 3-1: 7-Day average maximum asphalt temperatures

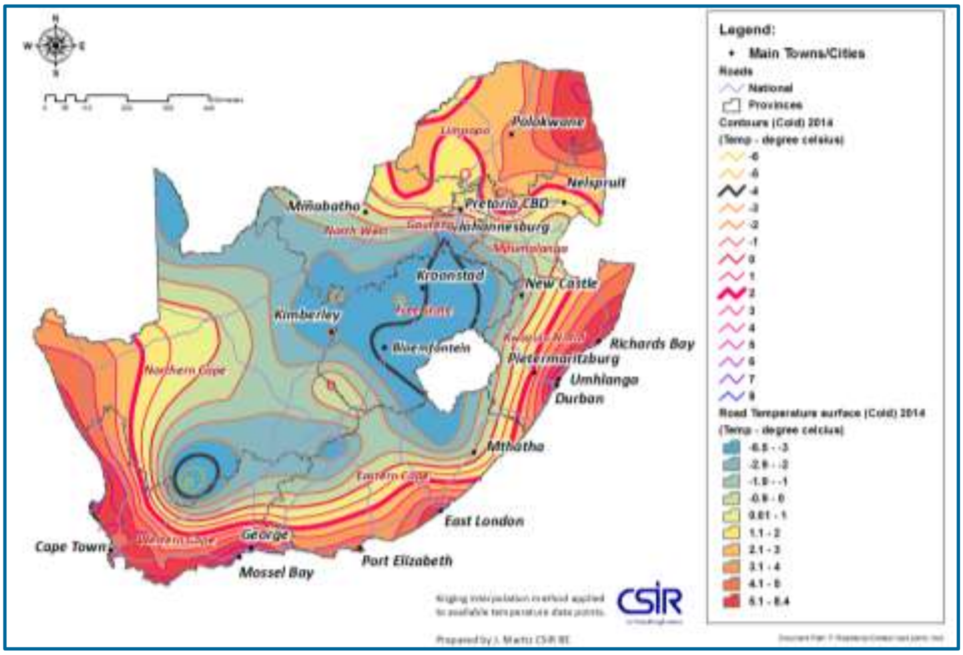


Figure 3-2: Minimum asphalt temperatures

It is proposed that a single low temperature grade of -10˚C for binders will suffice to cover the entire country and will simplify the number of binder grades required as well as minimise the logistics requirement in terms of the number of production requirement, storage tanks, etc. For practical purposes, South Africa can be considered to be covered by one intermediate service temperature, provisionally 25˚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 four levels of traffic loading be adopted, in terms of E80's and ruling speed.

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

< 10 million E80s
10 – 30 million E80s
> 30 million E80s

Design speeds fall within the following categories:

< 20 km/h
20 – 70 km/h
> 70 km/h

It is proposed that the combined effect of traffic loading and speed will be categorised as follows:

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

Classification of traffic in terms of loading intensity and speed is given in Table 8.

Table 8 Traffic classification

Traffic Volume (million ESAL)	Traffic Speed (Km/h)		
	< 20	20 - 70	> 70
< 10	H	S	S
10 – 30	V	H	H
> 30	E	V	V

The PG binder specifications for South Africa will be published under the auspices of the South African National Standards (SANS). However, the following classes and specification principles given in Table 9 and Table 10 will be maintained in the PG specification.

Note 3.1: Where compliance criteria are indicated in these Tables, the values are tentative at this stage.

Table 9: Classes for PG specification for asphalt binders – PG 64

Binder Class	Proposed specification			
	64S	64H	64V	64E
Original binder				
Maximum pavement design temperature (˚C)	64			
Non-recoverable compliance, J_{nr} at σ = xx kPa[1] @ 64 ˚C (kPa^{-1})				
Viscosity @ 135˚C (Pa.s)	≤ 3.0			
Flash Point (˚C)	≥ 230			
Storage stability @ 160˚C – Ratio of highest J_{nr} at σ = xx kPa[1] @ 64 ˚C to lowest (top and bottom)	≤ 1.5[2]			
RTFOT binder				
Mass change (m/m%)	|0.3| max			
J_{nr} at σ = xx kPa[1] @ 64 ˚C (kPa^{-1})				
PAV binder - @ yy˚C [3]				
Fatigue [4]	TBA[5]			
Thermal fracture [6]	TBA[7]			

[1]The stress level to be adopted in SA needs to be validated through testing and research.

[2] The storage stability specification limit is a preliminary value. A final value needs to be validated through testing and research.

[3] The ageing procedure to be adopted in SA is subject to further investigation

[4] A fatigue parameter has not been decided upon. The binder yield energy test (BYET) and the linear amplitude sweep (LAS) are possibilities to be investigated for possible specification parameters.

[5]A specification limit needs to be determined after testing and research.

[6] A low temperature thermal cracking parameter has not been decided upon.

[7] A specification limit needs to be determined after testing and research

Table 10: Classes of PG specification for asphalt binders – PG 58

Binder Class	Proposed specification			
	58S	58H	58V	58E
Original binder				
Maximum pavement design temperature (˚C)	≤ 58			
Non-recoverable compliance, J_{nr} at σ = xx kPa[1] @ 64 ˚C (kPa^{-1})				
Viscosity @ 135˚C (Pa.s)	≤ 3.0			
Flash Point (˚C)	≥ 230			
Storage stability @ 160˚C – Ratio of highest J_{nr} at σ = xx kPa[1] @ 64 ˚C to lowest (top and bottom)	≤ 1.5[2]			
RTFOT binder				
Mass change (m/m %)	|0.3| max			
J_{nr} at σ = xx kPa[1] @ 64 ˚C (kPa^{-1})				
PAV binder - @ 100˚C[3]				
Fatigue [4]	TBA[5]			
Thermal fracture [6]	TBA[7]			

[1] The stress level of 3.2 kPa is a preliminary value. A final value needs to be validated through testing and research.

[2] The storage stability specification limit is a preliminary value. A final value needs to be validated through testing and research.

[3] The ageing procedure to be adopted in SA is subject to further investigation

[4] A fatigue parameter has not been decided upon. The binder yield energy test (BYET) and the linear amplitude sweep (LAS) are possibilities to be investigated for possible specification parameters.

[5] A specification limit needs to be determined after testing and research.

[6] A low temperature thermal cracking parameter has not been decided upon.

[7] A specification limit needs to be determined after testing and research.

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 binder-blind and will promote cost effective use of costly modified binders, and
4. The specification is aligned to international practice.

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 3-1 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 according to Table 8.

3.3 Binder selection for specific mix types

Until such time as when a performance grade specification is fully implemented, binder selection would be based on the current specification - SANS 4001-BT1 and guidelines in AsAc TG1.

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

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 3.1: 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, they must be done so with the additives already present.

Note 3.2: 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.

3.3.7 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 3.3: 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 (4.75 mm) sieve[1];
- Fine aggregates (crusher sand, clean natural sand, mine sand, selected river gravel or a mixture of these.) – materials passing the 5 mm (4.75 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.

4.3.3 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).

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[1] 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.

4.3.3.1 Slag aggregates

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

Steel slag is a waste 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 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 a 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.

4.3.3.2 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 (2009).

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

4.3.4 Fillers

Fillers are essential for producing asphalt mixes which is 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 11 summarises filler types, characteristics and test methods to determine their properties.

Table 11: Filler types and characteristics

Type of filler/origin	Characteristics	Test method / Criteria
Hydrated lime (active filer)	• Improves adhesion between binder and aggregate • Improves mix durability by retarding oxidative hardening of the binder • Low bulk density and high surface area • Relatively high cost • Monitor effect on stiffness to ensure compactability	• Grading (% passing 0.075 mm) (SANS 3001-AG1): **minimum 70** • Bulk density in toluene (BS 812): **0.5 – 0.9 g/ml** • Voids in compacted filler (BS 812): **0.3 – 0.5%** • Methylene blue test (SANS 6243): **maximum value 5**
Portland cement (active filler)	• Relatively high cost • Monitor effect on stiffness to ensure compactability	
Baghouse fines	• Variable characteristics require control • Some source types may affect mix durability • Some types may render mixes sensitive to small variations in binder content	N/A
Limestone dust	• Manufactured under controlled conditions and complies with set grading requirements • More cost-effective than active filler • Although it is viewed as an inert filler, the high pH value reduces moisture susceptibility	N/A
Fly ash (non-active filler)	• Low bulk density • Relatively high cost • 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. 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., maximum ratio of approximately 1.6).

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 grading

In aggregate grading, a sample of aggregate materials 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. Typical gradings of various asphalt mix types were listed in Table 7 (Chapter 2). Table 12 shows the comparative sieve sizes for aggregate grading in South Africa. Sieve sizes as per SANS 3001 are used in this document.

Table 12: Changes in sieve sizes from TMH1 to SANS

TMH 1 sieve sizes [mm]	SANS 3001 sieve sizes [mm]
37.5	37.5
26.5	25
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 Grading requirements

4.5.1 Grading control points

To achieve suitable aggregates 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 maximum particle size (MPS) should be selected in accordance with Table 2, and is the smallest sieve size through which 100 percent of the aggregate particles passes),
- 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),
- The 2 mm sieve, and the 0.075 mm sieve.

Table 13 provides grading control points for four nominal maximum particles sizes of aggregates typically used for production of sand skeleton (often continuously graded) asphalt mixes in South Africa. The control points for 14 mm MPS are plotted in 0.45 power chart (Figure 4-1) for illustration purposes.

Note 4.6: The control points given in Table 13 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, needs to be employed.

Note 4.7: 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[1], which has been used with success in heavy duty asphalt applications in South Africa. In doing so, the designer should be mindful of the fact that some parameters of this method are based on aggregates encountered in the USA. Consequently its application in South Africa should be approached with some caution. It should be noted, though, that 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 13: Aggregate grading control points

Sieve sizes [mm]	Percent passing nominal maximum particle size (NMPS)							
	NMPS = 25 mm		NMPS = 20 mm		NMPS = 14mm		NMPS = 10 mm	
	Min	Max	Min	Max	Min	Max	Min	Max
37.5	100							
25	90	100	100					
20		90	90	100	100			
14				90	90	100	100	
10						90	90	100
7.1								90
5								
2	19	45	23	49	28	58	32	67
1								
0.6								
0.3								
0.15								
0.075	1	7	2	8	2	10	2	10

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

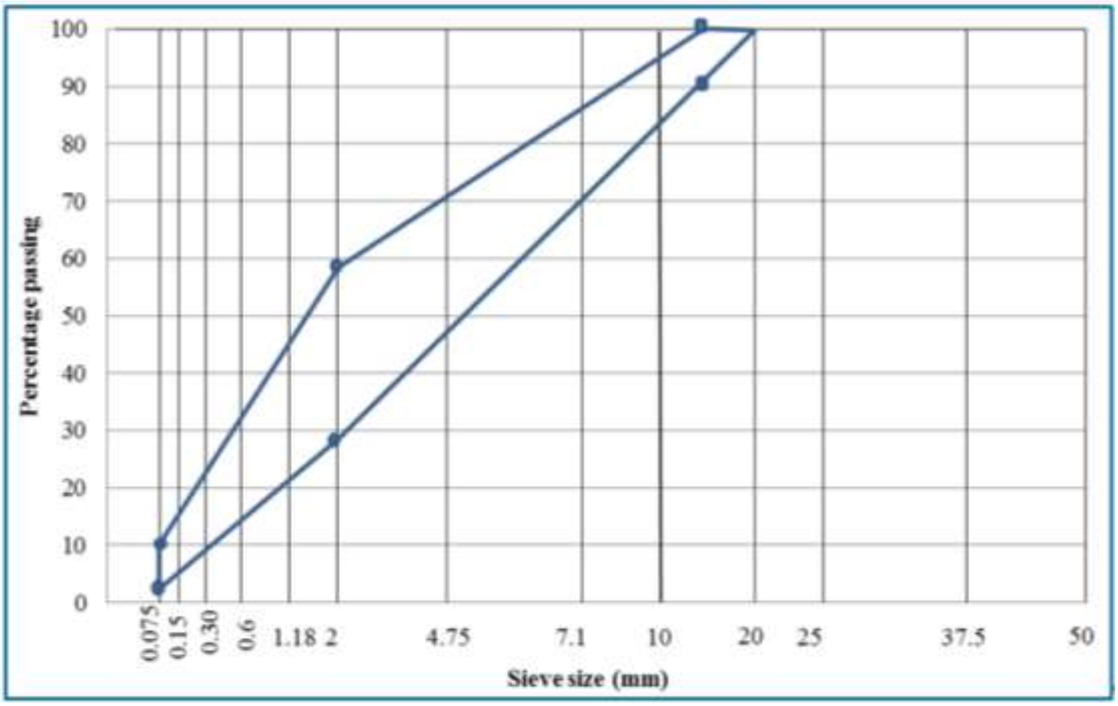

Figure 4-1: Grading control points plotted on 0.45 power chart for MPS = 14 mm

4.5.2 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 14 shows the percent passing control points of differentiation between coarse and fine mixes for various primary control sieves.

Table 14: Percent passing PCS control sieve

NMPS	PCS	PCS control point [% passing]
25 mm	5 mm	40%
20 mm	5 mm	47%
14 mm	2 mm	39%
10 mm	2 mm	47%

4.6 General requirements and specifications for aggregates

- 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 15.

4.7 Preparation and selection of aggregate grading

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

i. 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.

ii. 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.

iii. 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.

iv. Determine properties of individual aggregate fractions. The recommended test methods and criteria are presented in Table 15.

v. Combine the gradings of 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.

$$P = Aa + Bb + Cc, \ldots \tag{Eq. 4.1}$$

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

$A, B, C\ldots.$ = percentage of materials passing a given sieve for aggregates A, B, C

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

vi. Prepare a minimum of three trial aggregate blends; plot the grading of each trial blend on a 0.45-power chart, and compare the gradings of the trial blends with the guidelines provided in Table 13 (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 15: Recommended tests and criteria for aggregate selection

Property	Test	Standard	Criteria
Hardness / Toughness	Fines aggregate crushing test: 10% FACT	SANS 3001-AG10	Asphalt surfacings and base: minimum 160 kN Open-graded surfacings and SMA: 210 kN
	Aggregate crushing value (ACV)	SANS 3001-AG10	Fine graded: minimum 25% (Fine) Coarse graded: minimum 21%
Soundness	Magnesium sulphate soundness	SANS 5839 SANS 3001-AG12	12% to 20% is normally acceptable. Some specifications requires ≤ 12% loss after 5 cycles

Durability	Methylene blue adsorption indicator	SANS 6243	High quality filler: maximum value 5 More than 5: additional testing needed
Particle shape and texture	Flakiness index	SANS 3001- AG4	• 20 mm and 14 mm aggregate: maximum 25[1] • 10 mm and 7.1 mm aggregate: maximum 30
	Polished stone value (PSV)	SANS 3001–AG11	Minimum 50[2]
	Fractured faces	SANS 3001-AG4	• Fine graded: at least 50% of all particles should have three fractured faces • Coarse graded and SMA: at least 95% of the plus 5 mm fractions should have one fractured face
Water absorption	Coarse aggregate (> 5mm)	SANS 3001-AG20	Maximum 1% by mass
	Fine aggregate (< 5mm)	SANS 3001- AG21	Maximum 1.5% by mass
Cleanliness	Sand equivalency test	SANS 3001-AG5	Minimum 50 total fines fraction
	Clay lumps and friable Particles	ASTM C142–97	Maximum 1%

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

4.8 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 surface area (*SA*) of the aggregate particle is calculated based on Eq. 4.2:

$$SA = (2 + 0.02\,a + 0.04\,b + 0.08\,c + 0.1\,d + 0.30\,e + 0.60\,f + 1.6g) \times 0.20482 \qquad \text{(Eq. 4.2)}$$

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.

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 blending of aggregates and with 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.

- Mixes containing high percentage of coarse aggregates have the tendency to segregate and could be difficult to compact;
- Too high or too low filler in the mix can also affect workability;
- Too low or too high temperature will make the mix unworkable or tender, respectively.

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 40°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. High voids, which may accelerate binder ageing, or low binder content could lead to low fatigue life. 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. 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;
- Well compacted asphalt layers.

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, currently being formulated 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 Volumetric properties and definitions

Volumetric properties are defined in accordance with the schematic representation of the volume of compacted asphalt mix shown in Figure 5-1.

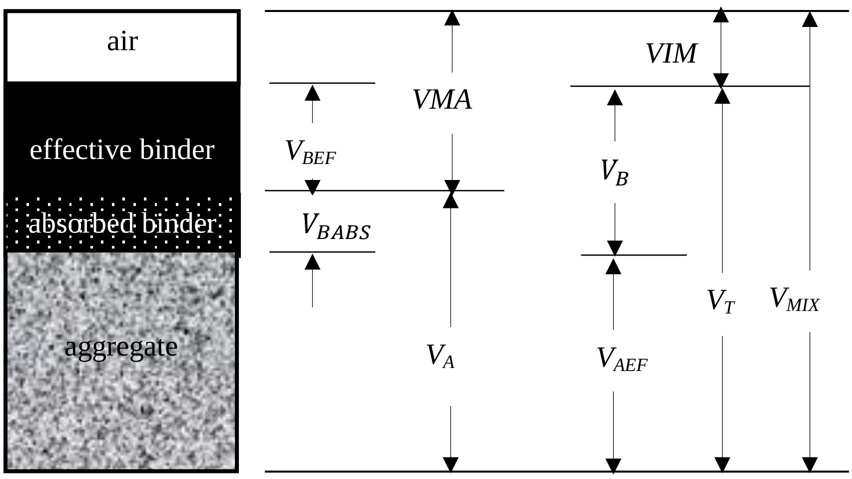

Figure 5-1: Volumetric parameters of compacted asphalt specimen

VIM = Volume of voids, represents the volume of the pores in the mix and interstices.

VMA = Volume of voids in mineral aggregate.

V_B	=	Total volume of binder within the asphalt mix.
V_{BABS}	=	Volume of absorbed binder that penetrates into the aggregate pores.
V_{BEF}	=	Effective volume of binder i.e. that which does not penetrate into aggregate pores.
V_A	=	Bulk volume of aggregate, including all permeable surface pores.
V_{AEF}	=	Effective volume of aggregate excluding surface pores filled with binder.
V_T	=	Total volume of binder and aggregate in the mix.
V_{MIX}	=	Total (apparent) volume of compacted asphalt specimen.

Table 16 and

Table 17 present important terminologies and test methods to determine various parameters of the components of asphalt mixes.

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) voids, but excluding the inter-particle voids, expressed in kilograms per cubic metre (kg/m³)	SANS 3001-AG20 (> 5 mm) SANS 3001-AG21 (< 5 mm)
Apparent density of aggregate	AD	Mass of the aggregate particles divided by the volume of the aggregate particles including impermeable (internal) voids but excluding permeable (surface) and inter-particle voids, expressed in kilograms per cubic metre (kg/m³)	SANS 3001-AG20 (> 5 mm) SANS 3001-AG21 (< 5 mm)
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) SANS 3001-AG21 (< 5 mm)
Bulk density of binder	BD_B	The bulk density of the binder, expressed in kilograms per cubic metre (kg/m³)	Method E2 (TMH1)
Bulk density of mix	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)	SANS 3001-AS10
Maximum void-less density of the mix (Rice method)	MVD	Mass per unit volume of a void-less bituminous mixture at a known test temperature	SANS 3001-AS11

Note 5.1: For the purpose of calculations, the bulk density of penetration grade binder may be taken as 1 020 kg/m³. Where modified binders are used obtain the bulk density of the binder from the supplier (SANS 3001-AS11).

Table 17: Volume parameters used in volumetric analysis

Parameter	Symbol	Definition	Formula
Voids in the mix	VIM	Difference between the MVD and the BD, expressed as a percentage of the MVD	$VIM = 100 \times \left[\frac{(MVD - BD_{Mix})}{MVD}\right]$
Binder content	M_B	Mass of binder in the mix, expressed in grams (g)	SANS 3001 - AS 11 SANS 3001 – AS1
	V_B	Volume of binder in the mix, expressed in cubic centimetres (cm³)	$V_B = \frac{1\,000 \times M_B}{BD_B}$
	P_B	Percentage of binder, expressed as a percentage of total mix	$P_B = 100 \times \left(\frac{M_B}{M_A + M_B}\right)$
Aggregate content	M_A	Mass of aggregate in the mix, expressed in grams (g)	SANS 3001 - AS 11 SANS 3001 – AS1
	V_A	Volume of the aggregate in the mix, expressed in cubic centimetres (cm³)	$V_A = \frac{1000 \times M_A}{BD_A}$
	P_A	Percentage of aggregate, expressed as a percentage of total mix	$P_A = 100 \times \left(\frac{M_A}{M_A + M_B}\right)$
Effective binder contents	V_{BEF}	Volume of effective binder expressed as a percentage of the volume of the bulk mix	$V_{BEF} = \frac{B_{EF} \times BD_{MIX}}{BD_B}$
	P_{BEF}	Percentage of effective binder in the mix (i.e. the total binder less the binder absorbed)	SANS 3001-AS11
Absorbed binder contents	M_{BABS}	Mass of the binder absorbed in the mix, expressed in grams (g)	SANS 3001-AS11
Volume of absorbed binder	V_{BABS}	Volume of binder absorbed into the pores (permeable voids) in the aggregate	$V_{BABS} = BD_{MIX} \times \left[\left(\frac{M_B}{BD_B}\right) + \left(\frac{M_A}{BD_A}\right) - \left(\frac{100}{MVD}\right)\right]$
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	$VMA = VIM + V_{BEF}$
Voids filled with binder	VFB	Percentage of voids in the bulk mix filled with binder	$VFB = 100 \times \left(\frac{V_{BEF}}{VMA}\right)$

5.5 Mix design levels

This manual presents three levels of mix design i.e., Level I, 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 volume (expressed as E80s) over the service life of the asphalt pavement and the risks associated with structural damage.

Figure 5-2 presents general recommendations for applying the three design levels.

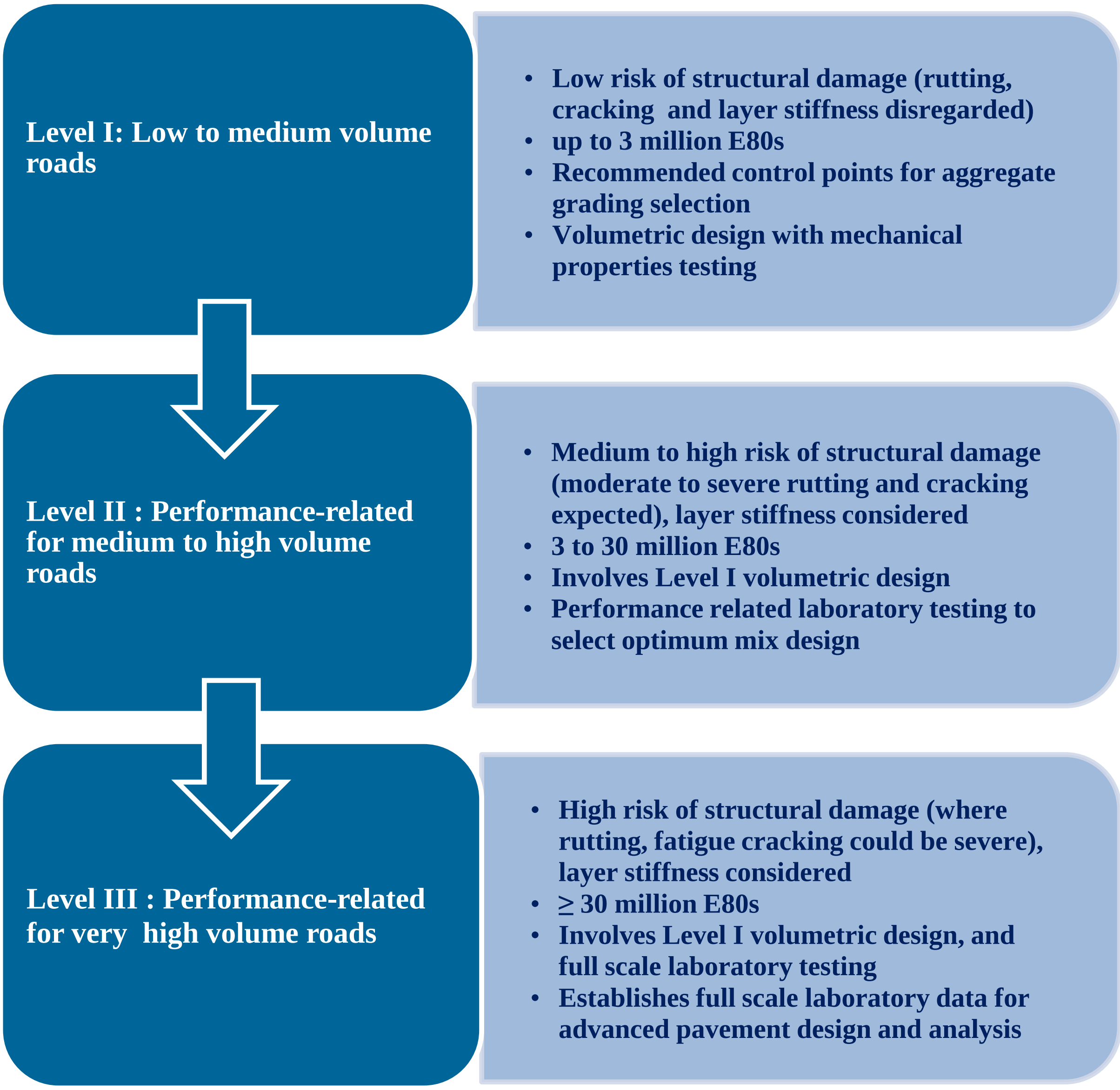

Figure 5-2: Mix design levels

5.5.1 Level I mix design process

The design process for Level I is shown in Figure 5-3.

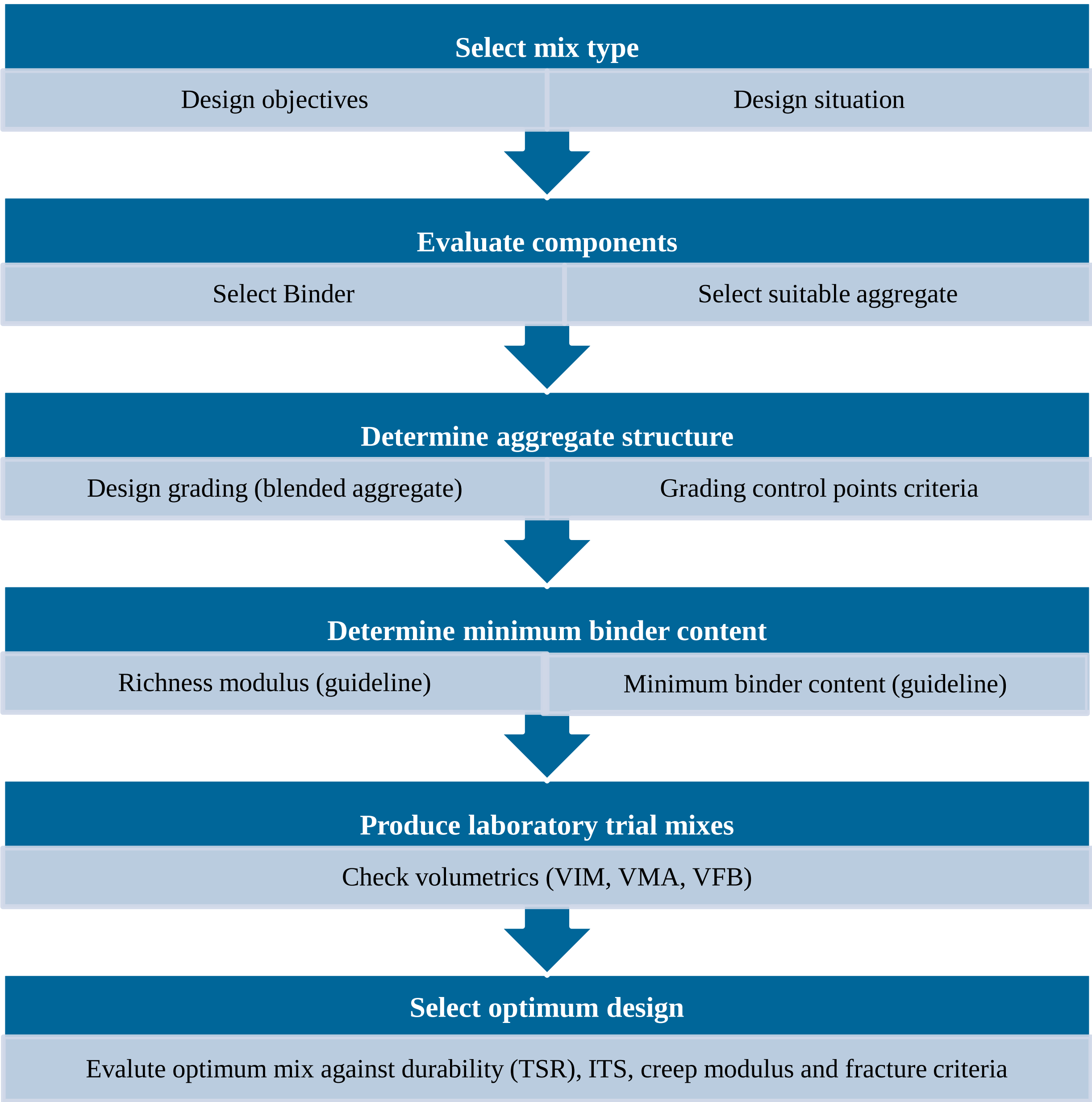

Figure 5-3: Level I design process

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 structure is established by:

(a) Determine minimum binder content for each trial blend using richness modulus, specific surface area and density of the aggregates. Richness modulus (*K*) is a measure of the binder film thickness surrounding the aggregate.

Eq. 5.1 yields the required minimum binder content based on these properties:

$$B_{PCC} = K \times \alpha \times \sqrt[5]{SA} \qquad \text{(Eq. 5.1)}$$

where:

B_{PCC} = mass of binder *expressed as a percentage of the total dry mass of aggregate, including filler*. B_{PCC} can be converted to the binder content by mass of total mix (P_B) generally used in South Africa using Eq. 5.2

$$B_{PCC} = \frac{100 \times P_B}{(100 - P_B)} \qquad \text{(Eq. 5.2)}$$

K = richness modulus - minimum K values for mix types evaluated for this manual are provided in Table 18.

α = correction coefficient for the density of the aggregate (BD_A), computed as follows:

$$\alpha = \frac{2.65}{BD_A}$$

SA = specific surface area (m²/kg) defined in section 4.7.

Table 18 Typical minimum richness modulus values

Mix type	Minimum *K*
Sand skeleton	**≥ 2.9**
Stone skeleton	**≥ 3.4**

Note 5.2: The *K* values in **Table 18** are intended as a point of departure for the determination of the minimum binder content.

Note 5.3: The expression for binder content is different from the conventional expression of binder content.

(b) Evaluate the three trial blends:

i. Marshall or Superpave gyratory compactions are optional choices for volumetrics. For each trial blend, compact the three duplicate specimens following Marshall (SANS 3001-AS1) or Superpave gyratory (AASHTO T 312) test procedure. Also, prepare two loose asphalt samples for determination of the maximum void-less density (MVD) of the mix using SANS 3001-AS11.

ii. 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. Typical values for SA mixes are provided in Table 19.

iii. 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.

iv. Compact specimens immediately after completion of short-term oven conditioning to the recommended number of blows (Marshall) or to N_{design} (Superpave - the number of gyrations at which the air voids content equal to 4 percent) in accordance to Table 20.

v. Determine the bulk density (BD_{MIX}) of the compacted specimens in accordance with SANS 3001-AS10. Use the BD_{MIX} and *MVD* results (average values for each trial binder content) to compute the volumetric properties (VIM, VMA, VFB) of the mix at N_{design}.

vi. 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 N_{design}.

Table 19: Typical mixing and compaction temperatures

Mix type	Binder type[1]	Mixing temperature [°C]	Compaction temperature [°C]
Sand skeleton	Pen grade (50/70)	150	135
	AP-1	155	145
	AE-2	160	145
Stone skeleton	Pen grade (35/50)	150	140
	AE-2	160	145
	AP-1	165	142
	AR-1	170	145

[1] These mixing and compaction temperatures may not necessarily be the optimum values for all modified binders; the manufacturer recommendation should be followed.

Table 20: Compaction requirements for Levels I

Marshall	Superpave
No. of blows	N_{design}
75/45[2]	75

[2] 75 blows on the first side + 45 blows on the reverse side

(5) 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. 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 – this is calculated as the percent by mass of the material passing the 0.075 mm sieve (based on wet sieve analysis) divided by the effective binder content.

iii. Prepare three duplicate specimens at each trial binder content. Specimens are prepared and compacted in the same manner as the specimens used to select the design aggregate grading.

iv. Determine the BD_{MIX} of all specimens. A minimum of two specimens are also prepared to determine the maximum void-less density (*MVD*) for each trial binder content.

v. Determine volumetric properties (VIM, VMA, VFB) of the compacted specimens.

vi. 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 22**.

vii. 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 that in-place voids is approximately equal to design voids +3%. Calculate the tensile strength ratio, and check results against the criteria presented in **Error! Reference source not found.**

viii. The requirements and criteria to attain the optimum design for Level I are given in Table 24.

ix. 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 21: Minimum percent VMA

NMPS (mm)	Minimum VMA[1] for design voids		
	3%	4%	5%
25	11	12	13
20	12	13	14
14	13	14	15
10	14	15	16

[1] Only values for continuously graded mixes are available and presented in this table.

Table 22: Percent VFB

Minimum	Maximum
65	75

Note 5.4: 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 5.5: The effect of grading on VMA is somewhat complex, however denser gradings generally lead to a decrease in VMA. Also larger aggregates (MPS) reduce VMA. Low VMA are very sensitive to slight changes in binder content. Generally, economising the binder content by lowering VMA is counter-productive and should be avoided.

Note 5.6: 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. Meeting VFB requirements avoids less durable mixes resulting from thin films of binder on the aggregate particles.

Note 5.7: 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 23: Moisture resistance criteria (Min TSR)

Climate	Permeability		
	Low	Medium	High
Dry	0.60	0.65	0.70
Medium	0.65	0.70	0.75
Wet	0.70	0.75	0.80

Note 5.8: 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 24: Summary of empirical performance tests for Level I

Property	Test	Method	Criteria
Durability/TSR	Modified Lottman	ASTM D 4867 M	See Table 23
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
Fatigue/tensile strength	Semi-circular bending (SCB)	BS EN 12697-44	@ 10°C (Criteria to be finalised)
Permeability	Water permeability	EN 12697-19[1]	0.1mm/s - 4 mm/s

[1]Method for determining permeability of asphalt mixes with interconnecting voids.

Note 5.9: The semi-circular bending test (SCB) is an optional parameter; it is recommended that it be carried out where layer configurations and stiffness's are such as may lead to fatigue distress in the asphalt layer.

Note 5.10: 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.

(6) 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.

5.5.2 Level II and Level III design process

The design process for Level II and Level III is shown in Figure 5-4. The volumetric design of Level I is the starting point for these levels. In comparison with 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.

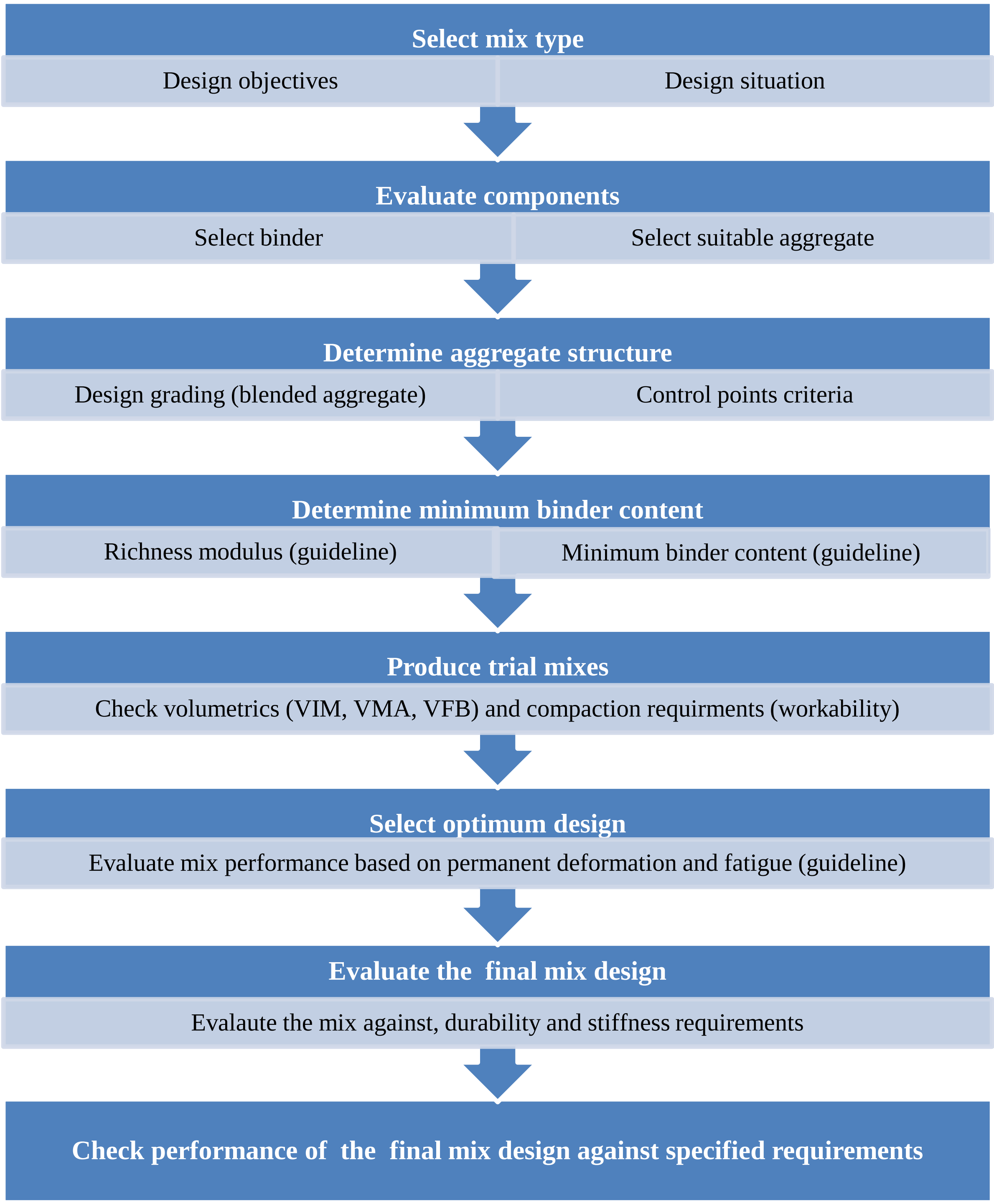

Figure 5-4: Level II and Level III mix design process

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 (AASHTO T 312) test procedure is used. Compaction and VFB requirements for Level II and Level III as presented in Table 25 and Table 26 are different.

Table 25: Laboratory compaction requirements for Levels II & III

Design traffic [E80s]	N_{design}
3 to 30 million	**100**
> 30 million	**125**

Table 26: Percent VFB (Heavy to very heavy traffic)

Design traffic [E80s]	Minimum	Maximum
3 to 30 million	**65**	**75**
> 30 million	**65**	**75**

2. Evaluate the mix against workability requirements provided in Table 27. The workability test is conducted on a short-term aged gyratory compacted specimens of dimensions 150 mm diameter by 170 mm high as per AASHTO PP 60 testing procedures.

Table 27: Workability criteria[1]

Mix type	Number of gyrations	Voids
Sand skeleton	**25**	$0 < V_{25} - V_{des} < 2$
Stone skeleton	**25**	$0 < V_{25} - V_{des} < 2$

[1] Interim, requiring lab validation tests. V_N = voids at number of gyrations; V_{des} = design voids.

3. Evaluate durability of the mix by using the Modified Lottman test procedures (ASTM D4867M), and check results against the criteria set in **Table 23**.
4. Evaluate stiffness (expressed as dynamic modulus) of the mix at in-place voids by using the asphalt mix performance tester (AMPT) procedures contained in AASHTO TP 79. Typical dynamic modulus values for SA asphalt mixes are provided in Table 28.

Table 28: Typical stiffness (dynamic modulus) values at 10 Hz (MPa)[1]

Mix type	Binder type[2]	Temperature (°C)				
		-5	5	20	40	55
Sand skeleton	**50/70**	**24 200**	**19 800**	**10 000**	**1 700**	**450**
	AP-1	**26 200**	**21 700**	**11 200**	**1 900**	**700**
	AE-2	**19 850**	**15 500**	**6 800**	**1 100**	**500**
Stone skeleton	**35/50**	**24 750**	**19 800**	**10 150**	**2 300**	**600**
	AE-2	**22 150**	**18 000**	**7 950**	**1 200**	**500**
	AP-1	**25 000**	**21 250**	**12 500**	**3 000**	**950**
	AR-1	**13 000**	**9 000**	**3 600**	**850**	**350**
	AR-1	**9 200**	**5 750**	**2 250**	**500**	**--[3]**

[1] Interim, requiring lab validation tests.

[2] The binder type refers to the empirical grades, as binders could not be test according to the incomplete performance graded specification as yet.

[3]Typical values will be incorporated when they become available.

Note 5.11: 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 six temperatures (-5, 5, 20, 40 and 55°C).

5. Select the optimum mix design based on performance
   - ***Permanent deformation*** – Three binder content levels should be used to evaluate permanent deformation of the mix. These levels include the optimum (binder content at 4% voids, volumetric design at Level I), optimum−0.5%, and optimum+0.5%. Permanent deformation is evaluated using repeated axial load flow number test. The standard test procedure to be followed is the AMPT described in AASHTO TP 79.
     i. Prepare three duplicate sets of gyratory compacted samples following AASHTO PP 60 procedure to the dimensions of 150 mm diameter by 170 mm high. Specimens for testing are cored and cut from the 150 mm diameter by 170 mm high samples to a final nominal dimension of 100 mm diameter by 150 mm high to achieve in-place voids. Total number of specimens for testing is nine (3 repeats @ the 3 binder contents).
     ii. For Level II design, apply a deviator stress of 483 kPa and confining pressure of 69 kPa on the specimen subjected to a haversine loading of 0.1 s and 0.9 s rest period and test the specimen at one test temperature of 55°C. Conduct the test until the flow point is reached or until 10 000 load cycles. The flow point represents failure of the specimen.
     iii. For Level III design, apply three deviator stress levels of 138, 276, and 483 kPa and confining pressure of 69 kPa and test the specimen at three test temperatures of 25, 40 and 55°C to record cumulative plastic strain at 20 000 load cycles.
     iv. The binder content that provides better resistance to permanent deformation (higher flow number) is selected as the design binder content.
     v. Typical flow numbers of SA mixes at two temperatures are provided in Table 29.

Table 29: Typical flow number (FN) (cycles)[1]

Mix type	Binder type[2]	Temperature (°C)	
		40	55
Sand skeleton	50/70	850	120
	AP-1	8 100	1 000
	AE-2	900	80
Stone skeleton	35/50	1 900	250
	AE-2	1 300	150
	AP-1	4 000 – 6 500	--[3]
	AR-1	700	50

[1] Flow number parameter is defined as the number of load pulses when the minimum rate of change in permanent (plastic) strain in the mix occurs during the repeated load test. Flow number is an indication of rutting. Typically, asphalt mixes with high flow number can be expected to exhibit better rutting performance than a mix with low flow number under the same conditions. The flow number values presented in this Table is based on applied deviator stress of 600 kPa with no confining stress

[2] Binder type refers to the empirical grades, as binders could not be tested according to the incomplete performance graded specification as yet.

[3]Typical values will be incorporated when they become available.

- ***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.
  i. Prepare slabs from compacted mix and cut the beams (400 mm long by 65 mm wide by 50 mm high) to conduct the fatigue test. Three duplicate specimens are prepared and tested at the design voids and design binder content.
  ii. 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 fatigue curve for the mix.
  iii. 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.
  iv. Fatigue life of the mix (number of repetitions to failure) is defined as the load cycle at which the specimen reaches 50% reduction in flexural stiffness relative to the initial stiffness i.e. the stiffness at the first 50 repetitions.
  v. Typical fatigue life values of SA mixes at 10 °C are provided in Table 30.

Table 30: Typical fatigue life values (no. of reps to 50% reduction of flexural stiffness)[1]

Mix type	Binder type[1]	Fatigue life $\times 10^6$ @10°C		
		200με	400με	600με
Sand skeleton	50/70	1.2	0.03	0.004
	AP-1	4.9	0.04	0.002
	AE-2	14.0	0.35	0.040
Stone skeleton	35/50	0.9	0.02	0.002
	AE-2	10.2	0.15	0.013
	AP-1	1.0	0.03	0.004
	AP-1 (SMA)	6.8	0.19	0.023
	AR-1	-- [2]	--[2]	0.313
	AR-1	9.5	0.40	0.063

[1] Interim, requiring lab validation tests..[1]Binder type refers to the empirical grades, as binders could not be tested according to the incomplete performance graded specification as yet. [2]Typical values will be incorporated when they become available.

[2]Test was not done.

6. Conduct water permeability test on the design mix in accordance with EN 12697-19 procedures and check results against the criteria presented in **Table 24**.

7. 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.

Note 5.12: 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 5.13: Although not a strict requirement, field performance of the mix can be verified by MMLS.

Table 31 lists test properties testing conditions, and the number of compacted specimens required to conduct laboratory test for Level II and Level III designs.

Table 31: Summary of performance-related tests

Property	Test conditions	No. of specimens	Test method
Workability	Superpave gyratory compactor, air voids after specified number of gyrations (Table 5-13)	3	ASTM D 6925
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 TP 79
Permanent deformation	AMPT permanent deformation at maximum of three stress levels and three temperatures.	3	AASHTO TP 79
Fatigue	Four-point beam fatigue test at maximum of three strain levels and three temperatures.	9	AASHTO T 321

5.6 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.

5.6.1 Cold mixes

Reference documents: Sabita Manuals 14, 21 and TG2 Interim guideline 2002.

5.6.2 Porous asphalt

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

5.6.3 Mixes for light traffic in residential areas

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

5.6.4 Warm mix asphalt

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

5.6.5 EME asphalt

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

5.6.6 Mixes with reclaimed asphalt

Reference document: TRH 21*: 2009 Hot mix asphalt recycling.* [1]

5.6.7 Stone mastic asphalt (SMA)

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

[1] Since extensive experience has been gained with mixes containing reclaimed asphalt subsequent to the publication of this document, the reader should note that it is not up-to-date in all respects.

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 2014. 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 determines 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, thicker asphalt layers (e.g. > 60 mm thick) are regarded as structural layers which will deflect less than thinner asphalt layers (e.g. < 60 mm thick) under traffic loading. 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 its 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.

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;
- Asphalt resilient response (dynamic modulus models);
  - Witczak predictive model.
  - Hirsch predictive model.
  - Laboratory-derived values.
- Asphalt damage models;
  - Permanent deformation (rutting) model.

– Fatigue cracking model.

Note 6.1: The Witczak and Hirsch predictive models are used as an alternative to dynamic modulus values obtained directly from laboratory testing.

6.3.1 Binder ageing model

The data obtained from the recovered binder should be used for the calibration of ageing models for the LTPP sections involved. Ageing can be represented as in **Error! Reference source not found.**.

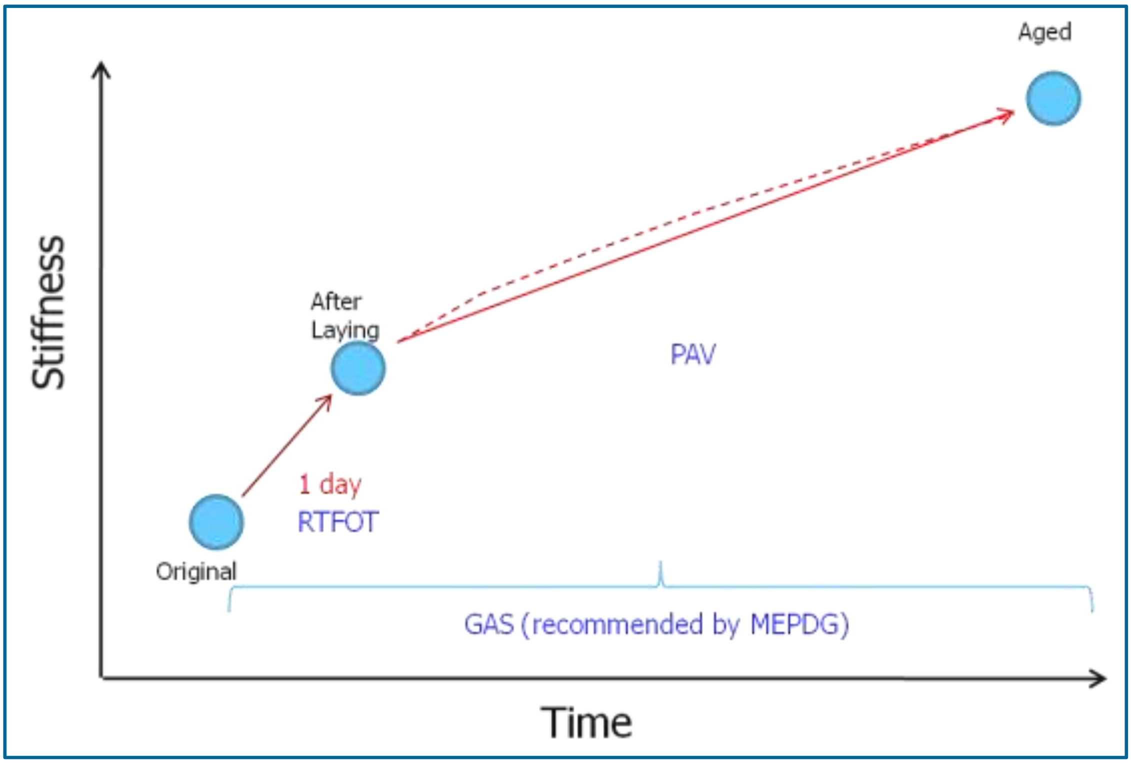

Figure 6-1: Pictorial presentation of ageing of asphalt

An interim ageing model (Denneman et al, 2011) is proposed in Eq. 6.1.

$$\eta_{aged} = \eta_{mod\,RTFOT} + \frac{t}{t_{PAV}} \times (\eta_{PAV} - \eta_{mod\,RTFOT}) \qquad \text{(Eq. 6.1)}$$

where:

η_{aged}	= viscosity after t months [Pa.s]
η_{modRTFOT}	= viscosity after modified RTFOT [Pa.s] – Represents mix/lay-down viscosity
η_{PAV}	= viscosity after t_{PAV} months [Pa.s]
t	= time at η_{aged} in months
t_{PAV}	= time presented by Pressure Aged Vessel (PAV) ageing and needs to be determined.

For un-modified binders, any convenient binder property pertaining to stiffness may be used and converted to viscosity, using the following conversion equations:

Penetration: converted to viscosity units using Eq. 6.2.

$\log \eta = 9.5012 - 2.2601 \log (Pen) + 0.00389 \log (Pen)^2$ (Eq. 6.2)

where:
η = viscosity, [Pa.s]
Pen = penetration for 100 g, 5 sec loading, 0.1mm

The softening point will yield a penetration of approximately 800 and a viscosity of 13 000 poise. Viscosity from the DSR data was calculated at temperatures ranging from 20˚C to 70˚C using Eq. 6.3 (NCHRP 1-37A, 2004).

$$\eta = \frac{G*}{10}\left(\frac{1}{sin\delta}\right)^{4.8628} \quad \text{(Eq. 6.3)}$$

where:
G^* = complex modulus of the binder at 1.59 Hz [kPa]
δ = phase angle [°]
η = viscosity [Pa.s]

Note 6.2: For modified binders, only dynamic shear rheometer (DSR) derived viscosity may be used.

6.3.2 Predicting dynamic modulus of asphalt

The SAPDM will use two predictive equations to determine dynamic modulus of asphalt:

- Witczak predictive equation, and
- Hirsch predictive equation.

6.3.2.1 Witczak predictive model

The Witczak predictive model for dynamic modulus of asphalt is shown in Eq. 6.4.

$$log\ \left|E^*\right| = 1.588582 + 0.029232\,P_{200} - 0.001767\left(P_{200}\right)^2 - 0.002841\,P_4 - 0.058097\,V_a$$
$$- 0.802208\frac{V_{beff}}{\left(V_{beff} + V_a\right)} + \frac{\left[3.871977 - 0.0021\,P_4 + 0.003958\,P_{38} - 0.000017\left(P_{38}\right)^2 + 0.00547\,P_{34}\right]}{1 + e^{\left(-0.603313 - 0.313351\,log\ f - 0.393532\,log\ \eta\right)}} \quad \text{(Eq.6.4)}$$

where:

$|E^*|$ = dynamic modulus, 10^6 [Pa] or [MPa]
η = bitumen viscosity, 10^5 [Pa.s]
f = loading frequency [Hz]
V_a = air void content [%]
V_{beff} = effective bitumen content, % by volume
$P_{3/4}$ = cumulative % retained on the ¾ in (19.0 mm) sieve

$P_{3/8}$ = cumulative % retained on the 3/8 in (9.5 mm) sieve
P_4 = cumulative % retained on the No. 4 (4.75 mm) sieve
P_{200} = % passing the No. 200 (0.075 mm) sieve

6.3.2.2 Hirsch predictive model

The Hirsch predictive equation (Eq.6.5) is an alternative to the Witczak model.

$$|E^*|_{mix} = 6.89476E\text{-}03 \times \left[P_c \left[4{,}200{,}000\left(1-\frac{VMA}{100}\right) + 4.35113E\text{-}04\,|G^*|_{binder}\left(\frac{VFA\ xVMA}{10{,}000}\right)\right] + \frac{1-P_c}{\left[\frac{\left(1-\frac{VMA}{100}\right)}{4{,}200{,}000} + \frac{VMA}{4.35113E\text{-}04 VFA\,|G^*|_{binder}}\right]}\right] \quad \text{(Eq.6.5)}$$

where:

$$P_c = \frac{\left(20 + \frac{VFA\,x\,4.35113E\text{-}04\,|G^*|_{binder}}{VMA}\right)^{0.58}}{650 + \left(\frac{VFA\,x\,4.35113E\text{-}04\,|G^*|_{binder}}{VMA}\right)^{0.58}}$$

$|E^*|$ = dynamic modulus [MPa]
$|G^*|_{\text{binder}}$ = shear complex modulus of binder [Pa]
VMA = per cent voids in mineral aggregates
VFB = per cent voids filled with binder
P_c = aggregate contact factor

Note 6.3: Both the Witczak and Hirsch dynamic modulus models are under investigation for final incorporation in the SAPDM. Until the investigation is completed either model can be used depending on available data. For example, if DSR data is available, then users are more likely to use the Hirsch model instead of the Witczak model.

6.3.2.3 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.

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

where:

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

f_r = reduced frequency [Hz]

δ = minimum value of $|E^*|$

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

β, γ = 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 6-2. 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).

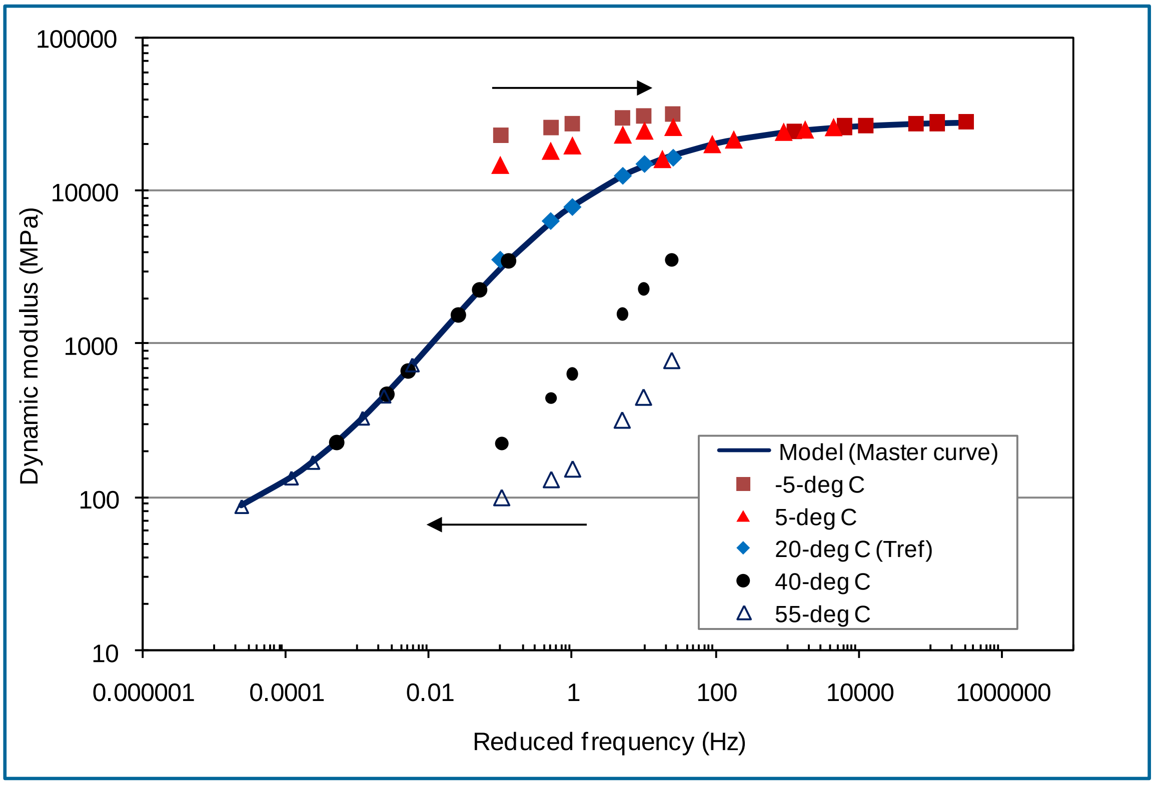

Figure 6-2: Typical master curve for dynamic modulus (Anochie-Boateng et al. 2011)

6.4 Predicting permanent deformation

The asphalt layer in the pavement is affected by temperature, stresses due to traffic loading and number of load applications. Based on repeated load triaxial testing procedures in the AMPT, these conditions were used to model permanent deformation of asphalt (Anochie-Boateng and Maina, 2012).

$$\varepsilon_p = k_1 \times N^{k2} T^{k3} \sigma_d^{k4} \quad \text{(Eq. 6.8)}$$

where:

ε_p = accumulated plastic strain; N = number of load repetitions; T = temperature [°C]

σ_d = applied deviator stress [kPa]; k_1, k_2, k_3, k_4 = nonlinear regressions constants

6.5 Predicting fatigue cracking

Currently, fatigue cracking in the SAPDM will require input data from ~~a~~ four-point beam testing described in chapter 5. The classical fatigue model for advanced pavement design is presented in Eq. 6.9.

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3} \quad \text{(Eq. 6.9)}$$

where;

N_f = number of repetitions to fatigue cracking

ε_t = tensile strain at the critical location

E = stiffness of the material

k_1, k_2, k_3 = nonlinear regressions constants

6.6 Temperature prediction models

6.6.1 Maximum surface temperature

The maximum temperature in the asphalt material within a pavement is estimated using Eq.6 which has been calibrated for South Africa climatic condition by Viljoen (2001).

$$T_{s(\max)} = T_{air(\max)} + 24.5(\cos Z_n)^2 \cdot C \quad \text{(Eq. 6.10)}$$

where:

$T_{s(\max)}$ = the daily maximum asphalt surface temperature in [ºC]

$T_{air(\max)}$ = the daily maximum air temperature in [ºC]

Zn = Zenith angle at midday

C = Cloud cover index

with:

C = 1.1 if $T_{air(\max)} > 30$ ºC

C = 1.0 if monthly mean air temperature < $T_{air(\max)}$ < 30 ºC
C = 0.25 if $T_{air(\max)}$ < monthly mean air temperature

The zenith angle is a function of the solar declination as shown in Eq. 6.11 below:

$$\cos(Z_n) = \sin(latitude)\sin(declination) + \cos(latitude)\cos(declination) \quad \text{(Eq. 6.11)}$$

For the purpose of this manual, an approximation of the solar declination is provided as Eq. 6.12. The ThermalPADS software contains a more accurate approximation of the daily solar declination.

$$declination = -23.45^{\circ}\cos\left[\frac{360^{\circ}}{365}\cdot(N+10)\right] \quad \text{(Eq. 6.12)}$$

where:
N = day of the year (with 1st of January = 1)

6.6.2 Minimum surface temperature

The algorithm by Viljoen (2001) provided in Eq. 6.13 is used to obtain the minimum surface temperature.

$$T_{s(\min)} = 0.89 \cdot T_{air(\min)} + 5.2 \quad \text{(Eq. 6.13)}$$

where:
$T_{s(\min)}$ = the daily minimum surface temperature [ºC]
$T_{air(\min)}$ = the daily minimum air temperature [ºC]

6.6.3 Asphalt temperature at depth

The prediction algorithm for maximum pavement temperature is provided in Eq.6.14.

$$T_{d\,(\max)} = T_{s\,(\max)}\left(1 - 4.237\times10^{-3}\,d + 2.95\times10^{-5}\,d^2 - 8.53\times10^{-8}\,d^3\right) \quad \text{(Eq. 6.14)}$$

where:
$T_{d(\max)}$ = Maximum daily asphalt temperature at depth d [ºC]
$T_{s(\max)}$ = Maximum daily asphalt surface temperature [ºC] from Eq. 6.10.
d = depth [mm]

The prediction algorithm for minimum pavement temperature at depth developed by Viljoen (2001) is shown as Eq. 6.15

$$T_{d(\min)} = \mathrm{T}_{s(\min)} + 3.7\times10^{-2}\,d - 6.29\times10^{-5}\,d^2 \quad \text{(Eq. 6.15)}$$

where:
$T_{d(min)}$ = Minimum daily asphalt temperature at depth d [ºC]
$T_{s(min)}$ = Minimum daily asphalt surface temperature [ºC] from Eq. 6.13.

Note 6.6:. Equation 6.16 may be used to obtain temperature variation in the asphalt layer during the day.

$$T_{d(t)} = T^{n}_{d(\min)} + \left[T_{d(\max)} - T^{n}_{d(\min)}) \right]^{\left[\frac{3.9(t-t_s)}{24-DL+1.5}\right]} \qquad \text{(Eq. 6.16)}$$

where:

t = hour t

t_s = time of sunset

$T^{n}_{d(\min)}$ = minimum temperature at depth d on the next day

$Td(t_s)$ = is temperature at sunset calculated using Eq. 6.15

$$DL = \frac{2}{15} \times \cos^{-1}\left[-\tan(latitude) \times \tan(solar\ declination) \right] \qquad \text{(Eq. 6.17)}$$

$$\alpha = 2 + \frac{d}{50} \qquad \text{(Eq. 6.18)}$$

6.6.4 Loading time

The relationship introduced by Brown (1973) can be used to calculate the loading time:

$$\log(t) = 0.5d - 0.2 - 0.94\log(v) \qquad \text{(Eq. 6.19)}$$

where:

t = loading time [s]

d = depth [m]

v = vehicle speed [km/h]

6.7 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 three 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 CONTROL, QUALITY ASSUARANCE AND ACCEPTANCE

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 control 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 quality control and acceptance control procedures required to ensure that the specification requirements of the asphalt mix are 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.

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 32, 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 32: Mix design levels

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

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, semi-circular bending, permeability and modified Lottman. Table 33 gives typical specification requirements for each parameter.

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, semi-circular bending, permeability and modified Lottman. This serves as a verification of the laboratory design.

Table 33: Level I design: Material, 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	BRD / ARD	Report Values
	Voids in Compacted Filler	Compliance with the requirements given in Table 11 and Table 15
	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 24
VFB	
ITS	
Dynamic creep	
Semi-circular Bending	
Permeability	
Modified Lottman (TSR)	
VMD	Report Only
BD_{MIX}	

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 34 shows the material properties and mix characteristics to be assessed, as well the permissible deviations.

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.

Table 34: 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. *Alternatively* ± 0.3% for continuous and semi-gap graded mixes, ± 0.4% for gap graded and bitumen rubber mixes
Grading (percentage passing sieve size)	Sieve size (mm)	
	25	±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 24
VFB		
ITS		
Dynamic creep		
Semi-circular Bending		
Permeability		
Modified Lottman (TSR)		
Compaction Density		The density shall be within the limits specified *Alternatively* (97% - Design voids ± 1%) of MVD

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 35. Layer thickness and levels are also monitored.

Table 35 Level I design: Permissible deviations from design/contract specifications at the paving stage as well as testing frequency

Property		Permissible deviation	Testing frequency
Binder content		The binder content shall be within the limits specified in the applicable statistical judgment scheme *Alternatively* ± 0.3% for continuous and semi-gap graded mixes, ± 0.4% for gap graded and bitumen rubber mixes	6 per lot [2]
Grading (percentage passing sieve size)	Sieve size (mm)		
	25	±5.0%	6 per lot [2]
	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%[1]	
VIM		± 1.5%	2 per lot[2]
Density/voids in mix		The density shall be within the limits specified in the applicable statistical judgment scheme *Alternatively* ± 1,5%	4 per lot [2]

Layer thickness	The layer thickness shall be within the limits specified in the applicable statistical judgment scheme	One day's work

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

[2] 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.

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 36.

Table 36: Material properties and mix characteristics to be certified

Property	Material property/Mix characteristic	Specification/certified/report values
Aggregate/filler	BRD / ARD	Report Values
	ACV	Compliance with specification requirement
	10% FACT	
	Magnesium Sulphate soundness	
	Methylene blue adsorption	
	FI	
	PSV	
	Fractured faces	

	Water absorption	As given in section 4.4 – 4.8
	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 37 shows permissible deviation of mix properties.

Table 37: Level II and Level III design: Permissible deviation from the certified values at the trial section

Property		Permissible deviation from certified values
Binder Grading/Type		Compliance with specification required
Binder content		The binder content shall be within the limits specified *Alternatively* ± 0.3% for continuous and semi-gap graded mixes, ± 0.4% for gap graded and bitumen rubber mixes
Grading (percentage passing sieve size)	Sieve size (mm)	
	25	±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% [1]
Design voids @ N_{design} (compacted loose mix)		Design value ± 1.5%
Density of the paved mix		The density shall be within the limits specified *Alternatively* (97% - Design voids ± 1,5%) of MVD

[1] 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 38.

Table 38: 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 content		The binder content shall be within the limits. *Alternatively* ± 0.3% for continuous and semi-gap graded mixes ± 0.4% for gap graded and bitumen rubber mixes	6 per lot [2]
Grading (percentage passing sieve	Sieve size (mm)		
	25	±5.0%	6 per lot[2]
	20	±5.0%	

size)	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%[1]	
Density of the paved mix		The density shall be within the limits specified *Alternatively* (97% - Design voids ± 1%) of MVD	4 per lot [2]
Layer thickness		The layer thickness shall be within the limits specified in the applicable statistical judgment scheme	One day's work

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

7.6 Test methods

Table 39 presents the list of test methods for evaluation of material properties, mix characteristics and performance-related parameters.

Table 39: 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 relative density and water absorption	SANS 3001-AG20
	Fine aggregate bulk density, apparent relative 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 3001-6243
	Clay lumps and friable Particles	ASTM C1426
	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 TP 79
	Fatigue	AASHTO T 321
	Permanent deformation	AASHTO T 312
	Workability	ASTM D 6925
	Durability	ASTM D4867M
	ITS	ASTM 6931
	Dynamic creep modulus	CSIR RMT-004
	Permeability	EN 12697-19

7.7 Asphalt paving and construction factors affecting quality control

7.7.1 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.

7.7.2 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.

7.7.3 Segregation

It is important to ensure that segregation of the mix does not occur. Segregation results in variability of mix composition 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.

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APPENDIX A

Overview of the Bailey method for determining aggregate proportions

While it has been noted in *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.

Aggregate grading

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

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. 4.2).
- 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.

$$PCS = 0.22 \times NMPS \qquad \text{(Eq. 4.2)}$$

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.

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 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 • Start of particle-to-particle contact • Determine LUW (kg/m³) • Determine volume of voids	AASHTO T19	V_{LUW} [1] **: 43% – 48%** $V_{LUW} = 100 \times \left[\frac{BD_A - LUW}{BD_A} \right]$
Rodded unit weight (RUW)	• Requires compactive effort ○ Three layers	AASHTO T19	V_{RUW} [2]**: 37% – 43%**

	○ Rodded 25 times each • Increased particle-to-particle contact • Determine RUW (kg/m³) • Determine volume of voids		$V_{RUW} = 100 \times \left[\frac{BD_A - RUW}{BD_A} \right]$
Chosen unit weight (CUW) (**Table 4-5**)	• Value that the designer selects based on the desired interlock of coarse aggregate • The designer must decide the desired mix type; fine-graded, coarse-graded or a stone mastic mix • 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**	**< 90**
Coarse-graded	**CA LUW**	**95 to 105**
SMA	**CA RUW**	**110 to 125**

CA = Coarse aggregate.

Note 4.3: 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.

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. 4.3. 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. 4.4. Typical ranges of voids are presented in

Table 42.

$$V_{LUW} = 100 \times \left[\frac{RDA - LUW}{RDA} \right] \qquad \text{(Eq. 4.3)}$$

$$V_{RUW} = 100 \times \left[\frac{RDA - RUW}{RDA} \right] \qquad \text{(Eq. 4.4)}$$

where~~,~~:
V_{LUW} = Loose unit weight voids
V_{RUW} = Loose 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 the mix types

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

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.

$$CA\ ratio = \frac{\text{Percentage passing half sieve} - \text{Percentage passing PCS}}{100 - \text{Percentage passing half sieve}} \qquad \text{(Eq. 4.5)}$$

$$FA_c\ ratio = \frac{\text{Percentage passing SCS}}{\text{Percentage passing PCS}} \qquad \text{(Eq. 4.6)}$$

$$FA_f\ ratio = \frac{\text{Percentage passing TCS}}{\text{Percentage passing SCS}} \qquad \text{(Eq. 4.7)}$$

Table 44 to **Table 48** show the control sieves and recommended aggregate ratios for fine-graded, coarse graded and SMA mixes.

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**
25	**5**	**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	2	1	0.6	0.15	--[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	5	2
25	14	5	2	1
20	10	5	2	1
14	7,1	2	1	0.3
10	5	2	1	0.3
7,1	5	2	1	0.3
5	2	1	0.3	0.15

Table 46: Control sieves for SMA mixes

(NMPS, mm)	Half sieve	PCS	SCS	TCS
20	10	5	2	1
14	7,1	2	1	0.3
10	5	2	1	0.3
7,1	5	2	1	0.3
5	2	1	0.3	0.15

Note 4.4: 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.

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

NMPS (mm)	CA (coarse-graded)	CA (fine-graded)	Coarse and fine -graded	
			FA_c	FA_f
37,5	0.80–0.95	0.60-1.00	0.35–0.50	0.35–0.50
25	0.70-0.85			
20	0.60-0.75			
14	0.50-0.65			
10	0.40-0.55			
7,1	0.30-0.50			
5	0.30-0.45			

[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	0.60-0.85
10	0.15-0.30	0.60-0.85	0.60-0.85

Note 4.5: The SANS sieves have come to effect in 2013. There is therefore, a need to review the Bailey concepts based on the new sieves to incorporate new aggregate ratios in this manual.

Effects of aggregate ratios on VMA

Tables 4-7 to 4-11 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 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 on VMA – Increasing 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 4.6: 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 4.7: Changes in the new ratios for fine-graded mixes create similar results in regards to the VMA.

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 41**). 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 voids in each fine aggregate according to its corresponding CUW and contribution by volume. Then sum the voids contributed by each fine aggregate.
7. Determine the chosen unit weight for the total aggregate blend (*contributions of coarse and fine fractions, % by volume*).
8. Determine the initial blend percentage by weight of each aggregate. Divide the unit weight of each aggregate fraction by the unit weight of the total aggregate blend.
9. Determine the amount of material passing 0,075 mm sieve contributed by each aggregate fraction.
10. Determine the amount of filler required, if any, to bring the percent passing the 0,075 mm sieve to the desired level.
11. Once the desired amount of material passing 0,075 mm sieve is achieved, adjust the final blend percentages (by volume) of fine aggregate fractions. In this step the blend percentage of coarse aggregate is not changed.
12. The final blending percentages (by mass) and aggregate ratios are determined and checked against Bailey requirements.

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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 B 1 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.

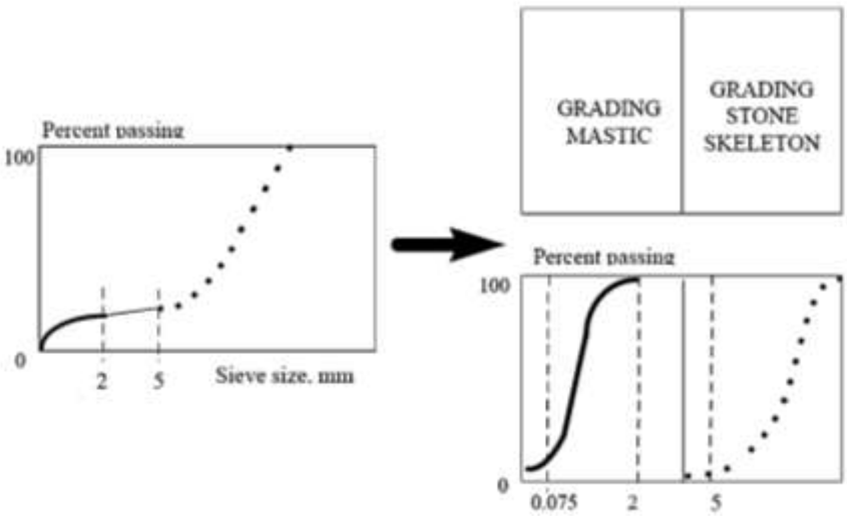

Figure B 1: Mix gradation components

B3.1 Design of Stone Skeleton

Based on the layer thickness to be used for the SMA surfacing a coarse aggregate (>2 mm) 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 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 B 2 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.

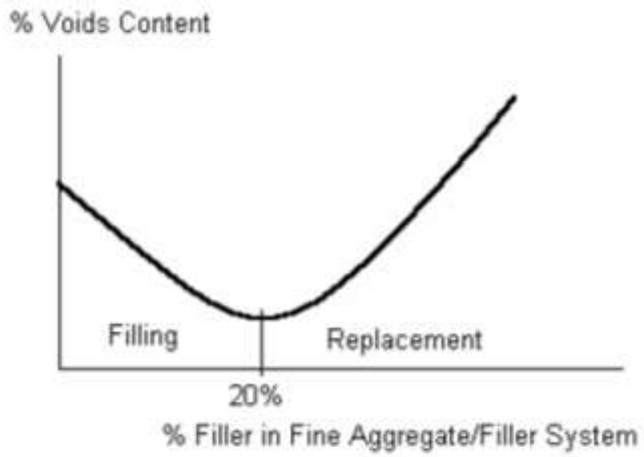

Figure B 2: Influence of Fine Aggregate : Filler Ratio

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 B 3 shows the relationship between voids and changing the coarse aggregate fraction while keeping the bitumen content constant.

Figure B 3: 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 B 4 the coarse aggregate (> 2mm) should be at least 69%.

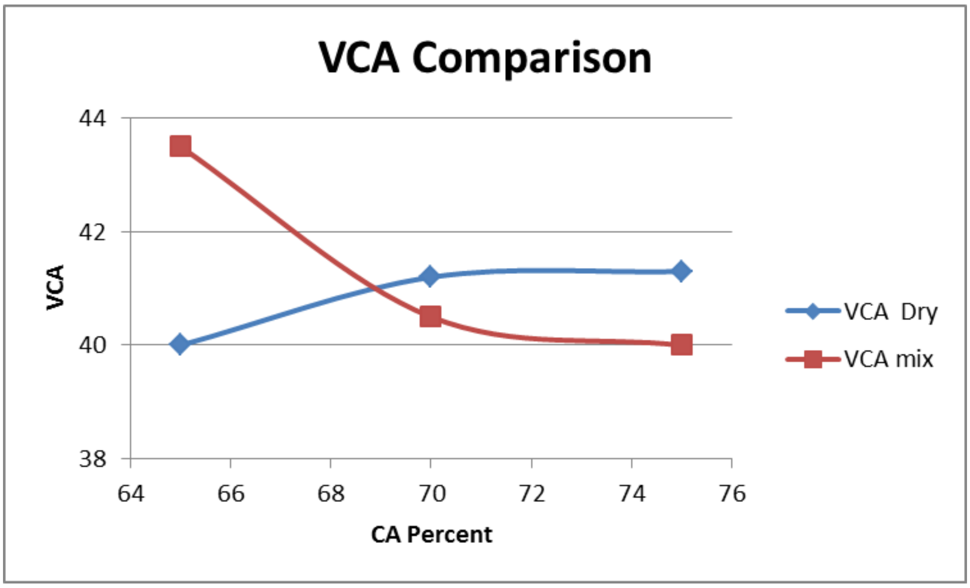

Figure B 4: Comparison of VCA$_{\mathbf{Dry\ and}}$ VCA$_{\mathbf{Mix}}$

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.