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Design and Use of Asphalt in Road Pavements

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DVD 420	Treatment Of Bitumen Burns
DVD 430	Working Safely with Bitumen
DVD 440	Firefighting In the Bituminous Products Industry
DVD 450	Safe Loading and Off-Loading of Bitumen
(*The following manuals have been withdrawn: 3, 4, 6, 9, 11, 14, 15, 16 and 21*)	

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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.

This manual is based largely on research commissioned by Sabita and carried out by the CSIR 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.

Since February 2020 several revisions have been incorporated, based on new research, the experiences in practice and keeping abreast of international best practice.

In this, the seventh edition, the scope of the document was explicitly defined as pertaining to the design of dense asphalt, with fine or coarse continuous gradings and produced with conventional or modified bitumen, where such modification agents result in a homogeneous binder, as distinct from non-homogeneous binders where the modifying agent is e.g. crumbed rubber. The design of special mixes should be carried out in accordance with the relevant Sabita manuals or, where the design of a mix type is not covered by the scope of a Sabita manual, by reference to best practice documents. Also, the chapter on quality management procedures has been removed for incorporation into Sabita Manual 5: *Guidelines for the Manufacture and Construction of Asphalt*.

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List of abbreviations

Abbreviation	Definition
AASHTO	American Association of State Highway and Transportation Officials
AMPT	Asphalt Mixture Performance Tester
AD	Apparent Density
A-E	Predominantly elastomer modified binder for asphalt
A-P	Predominantly plastomer modified binder for asphalt
A-H	Predominantly hydrocarbon modified binder for asphalt
A-R	Rubber modified binder for asphalt
ASTM	American Society for Testing and Materials
BD	Bulk Density
COLTO	Committee of Local Transport Officials
COTO	Committee of Transport Officials
DSR	Dynamic Shear Rheometer
E80	Equivalent 80 kN axle load
EME	Enrobé à Module Élevé
EN	European Standard / Europäische Norm
EVA	Ethylene Vinyl Acetate
FBR	Filler / binder ratio
IGDHMA	Interim Guidelines for the Design of Hot-Mix Asphalt
ITS	Indirect Tensile Strength
LTPP	Long-Term Pavement Performance
MEPDG	Mechanistic-Empirical Pavement Design Guide
MPD	Mean Profile Depth
MVD	Maximum voidless density
NMPS	Nominal Maximum Particle Size
PCS	Primary Control Sieve
QC / QA	Quality Control / Quality Assurance
RA	Reclaimed asphalt
RET	Reactive Elastomeric Terpolymer
RTFOT	Rolling Thin Film Oven Test
SANAS	South African National Accreditation System
SANRAL	South African National Road Agency SOC Ltd
SANS	South African National Standards
SAPDM	South African Pavement Design Method
SATAS	The South African Technical Auditing Servies
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
VCA	Voids in the Coarse Aggregate
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 method applicable to conventional hot-mix asphalt, i.e. dense asphalt with fine or coarse continuous gradings and made with conventional bitumen or homogeneous modified binders. This manual does not give detailed guidance on the design of special mixes (e.g., mixes produced at lower temperatures known as warm mix asphalts), mixes with non-homogeneous binders (e.g. bitumen-rubber), Enrobé à Module Élevé (EME) asphalt, stone mastic asphalt, porous asphalt, mixes for light traffic in residential areas, and mixes with reclaimed asphalt. For guidance on the design of these mixes the reader should consult the relevant Sabita Manual or specific best practice documentation

All mixes are grouped into sand-skeleton or stone-skeleton categories based on their aggregate packing characteristics and, hence, gradings. Whereas stone-skeleton mixes will largely fall within the domain of special mixes like porous asphalt, stone mastic asphalt and some mixes made with bitumen-rubber, conventional mixes, in general, comprise sand-skeleton types.

The information contained in this manual has been compiled from knowledge and experience recorded by the local asphalt industry and experienced practitioners, 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 a suitable binder and aggregate, which is workable in the field, to ensure:

- Satisfactory durability
- Fatigue performance
- Resistance to flushing and permanent deformation through the selection of suitable aggregate configurations providing structure and space between particles to accommodate sufficient binder
- A waterproof wearing course layer (with the exception of porous asphalt), meeting functional requirements such as friction, noise attenuation and comfort
- A base course layer meeting structural requirements over its design life

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.

1.3 Simplification

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

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

1.4 Design approach

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

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. They are meant to be guidelines to develop the aggregate grading, rather than strict specifications. This distinction provides the designer with additional flexibility in adjusting aggregate gradings to meet volumetric requirements of the mix. The Bailey method, which has been used with success in South Africa, can be used to optimise aggregate grading and mix design criteria. For this reason, a brief overview of the fundamental principles of the method is presented in Appendix A and the judicious application of the method merits serious consideration.

1.5 Link to pavement design

One of the shortcomings of the asphalt design methods previously available to South African practice was the lack of an explicit relationship between performance characteristics to meet established structural design requirements and consequent laboratory test targets to be achieved during the mix design process.

While considerable progress has been achieved in setting design goals in terms of targeting performance related properties, response and damage modelling in terms of dynamic modulus, and the resistance to fatigue and permanent deformation distress is not completely covered. To achieve such a situation performance related parameters should either be nominated by the structural designer or be derived from a nationally accredited standard pavement design procedure thereby ensuring that mixes will perform adequately in a range of applications, in terms of e.g. traffic and climate. Such a situation would constitute a "Balanced Mix Design" approach – a goal that is currently on the agenda of the Sabita TDFP working group reviewing this manual.

1.6 Certification of standard mixes

Where appropriate, mixes can be certified through agencies such as the South African Technical Auditing Services (SATAS) after a process of comprehensive performance related testing, evaluation of trial sections and examination of the pertinent quality assurance programme in place. The certification will be associated with specific plant setups, component material quality (aggregate, filler and binder), and mix characteristics, in respect of both component materials and relevant performance characteristics. 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 significantly.

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 and comply with the specification requirements. It is expected that the introduction of the performance-related mix design method will see the increased use of standard mix designs by producers and a reduction in the number of project specific mix designs.

In the European market it has become possible to get European Conformity (CE) markings for bituminous mixes, indicating that a product is fit-for-purpose. The CE certificate discloses the product's performance in various performance-related tests. In South Africa, a similar system could be considered 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.

1.7 Scope and structure of the manual

While the topic of asphalt mix design is covered broadly in this manual, detailed design procedures presented in the main body cover the "general case" – i***.e. asphalt made with generally available conventional bitumen or homogeneous modified binders***. Mix types would typically have continuous gradings and, as is currently the case, comprise mostly sand-skeleton types.

While noting that there are common factors that cover the design of a range of mix types, more detailed and specific mix design processes and procedures for the following special mixes are provided in various Sabita manuals, as set out in section 5.7 *Design of special mixes*.

- Enrobé à Module Élevé" (EME)
- Sand asphalt mixes
- Asphalt for lightly trafficked roads in residential areas
- Porous asphalt mixes
- Bitumen rubber asphalt
- Warm mix asphalt
- Reclaimed Asphalt

The provisions of these specific methods and procedures should be applied rigorously, even in cases where such provisions are divergent from those in this manual.

Note 1.1: At the time of the publication of this edition there were no Sabita manuals covering the design of sand-skeleton gap– or semi-gap graded mixes, as these mix types are not in general use. Also, there is no Sabita manual comprehensively covering the design of stone mastic asphalt (SMA). However, some guidance on the aggregate proportioning and reference to comprehensive design methods (e.g. AASHTO R46: *Standard Practice for Designing Stone Matrix Asphalt (SMA) using the Superpave Gyratory Compactor™ (SGC)* and AASHTO M 325-08 (2021): *Standard Specification for Stone Matrix Asphalt (SMA)* are given in Appendix B.

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

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

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

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

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

2. Mix type selection

2.1 Asphalt mix types

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

2.1.1 Sand-skeleton mixes

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

2.1.2 Stone-skeleton mixes

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

2.2 Factors impacting on selection of asphalt type

2.2.1 Traffic considerations

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

Heavy vehicles

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

Table 1: Traffic classification

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

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

i. Axle loads

Axle loads are limited to certain maximum values by law. The value of 80 kN is currently used as the basis for determining the equivalent single axle loading on the pavement.

ii. Traffic speed

The speed of heavy vehicles may significantly influence the performance of an asphalt mix. At *high speeds* the impact of the load on the pavement system is resisted not only by the combined stiffness of the pavement layers, but also by the inertial and damping forces generated within the pavement structure. These resisting forces will increase with vehicle speed, with a resultant reduction in the amount of deflection and bending which takes place in the asphalt layer. Dynamic pavement models as well as strain measurements taken at various vehicle speeds have shown that tensile strains at the bottom of the asphalt layer may decrease by as much as 50 % as vehicle speeds increase from crawl 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 non-recoverable strain, hence permanent deformation e.g. rutting in the wheel tracks. Mixes designed for climbing lanes, intersections, or any other condition where heavy vehicle speeds are predominantly less than approximately 30 km per hour require special consideration.

iii. Tyres

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

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

Light vehicles

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

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

2.2.2 Braking and traction

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

2.2.3 Fuel spillage

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

2.2.4 Wander

The degree of wander in the traffic lane can have a significant effect on rutting and fatigue. Wander is normally greater on lanes which are wide and have fast-moving traffic than on narrow lanes with slowly moving heavy traffic e.g. on dedicated bus routes. In the latter situation, the degree of channelization is increased, and consequently rutting resistance of the mix should be commensurate with the increased concentration of loading.

2.2.5 Layer or lift thickness and maximum particle size

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

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

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

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

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

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

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

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

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

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

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

2.2.6 Climatic considerations

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

Maximum temperature

Temperature is a key determinant for permanent deformation (rutting) potential of an asphalt layer. Maximum temperature will influence the selection of mix type, aggregate type, and binder type.

Intermediate and minimum temperatures

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

Temperature differentials

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

Rainfall

Mixes located in high rainfall areas or in areas with a large number of rainy days have an increased potential for stripping and may require special attention to be paid to durability issues. Such mixes may also have greater waterproofing requirements, depending on the underlying layers and therefore permeability may become an important design consideration. Rainfall patterns may thus influence the choice of aggregate type, filler type, and binder type.

2.2.7 Other considerations

Functional requirements

Special functional requirements may include:

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

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

Table 3 defines classes of texture and their characteristics.

Table 3: Classes of surface texture

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

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

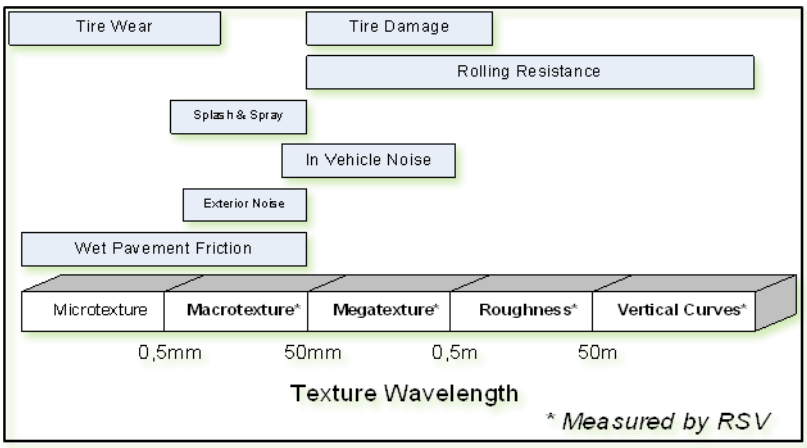

Figure 1: Functional requirements in relation to surface texture

Geometric conditions

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

Material availability and project specifications

- The availability of aggregates, filler and bitumen of the required quality should be evaluated before project specifications are finalised. Such evaluation at an early stage may lead to

innovative practice in the interest of cost-effectiveness or may alert the client and tenderer to additional costs that may be incurred through transport or special manufacturing processes required to produce the desired quality of materials in the mix;

- The designer should ensure that component materials available from particular sources are of adequate supply and can meet the project and product specifications. Materials should preferably be obtained from a fixed commercial source. The properties of a material product supplied should not vary significantly during the supply period. In addition, the quality of the products should be such that it will not be negatively affected by transportation to site;
- Situations in which the standard specifications are modified to suit the needs of the project require special attention to be paid to availability and properties of local materials. Designers should alert tenderers to non-standard project specifications that may have an impact on material availability, especially situations in which locally available materials do not meet the project specifications;
- The decision to procure a material from a particular source depends on factors such as location of the source in the project proximity, availability of the required materials (in quality and quantity) from the source, as well as the economic consequences to the project;
- In some cases, to promote equitable tendering, the client is well advised to indicate nominal proportions of component materials, e.g. bitumen, filler and aggregates based on preliminary mix designs.

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

Table 4: Location of aggregates used in asphalt

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

Note 2.1: Certain mix types 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.

At the time of publication of this edition bitumen for asphalt manufacture is available from a single SA refinery, importers and secondary producers. Bituminous binders are classified in terms of the mandatory national specification, SANS 4001-BT1, for conventional bitumen graded in terms of its penetration. Also, the nomenclature for modified binders as set out in TG1: *The Use of Modified Bituminous Binders in Road Construction* (November 2020) is still in use. As the implementation of

the provisions of SATS 3208 or a performance grade national specification takes effect, so the use of these categories would diminish until PG classification of binders will be implemented for general use.

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

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

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

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

2.3 Mix design considerations and mix type selection

The determination of aggregate packing characteristics of the mix (i.e. 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 with a coarse design gradings may be considered in cases where adequate skid resistance is an important consideration.
- To ensure adequate skid resistance of gap-graded and semi gap-graded sand-skeleton asphalt wearing courses, pre-coated chippings are usually spread on the freshly paved, hot mat prior to embedment by rolling;

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

- The practice of rolled-in-chips on continuously graded, dense asphalt is not recommended in view of the possible adverse effect on mix performance in terms of durability and permeability.

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

3. Binder selection

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

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

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

3.1 PG binder classification system

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

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

3.1.1 Temperature

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

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

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

- intermediate temperatures for fatigue (durability) and
- low temperatures for non-load associated fracture.

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

[d] In the light of forward projections of climate effects in SA in the paper by Mokoena et al on *The use of bias-corrected climate model projections for bituminous binder selection for the construction of resilient asphalt roads*, published in the Journal of the SAICE, June 2024, it is likely that these maps will be revised in SATS 3208 and, consequently, in SANS 4001-BT10.

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

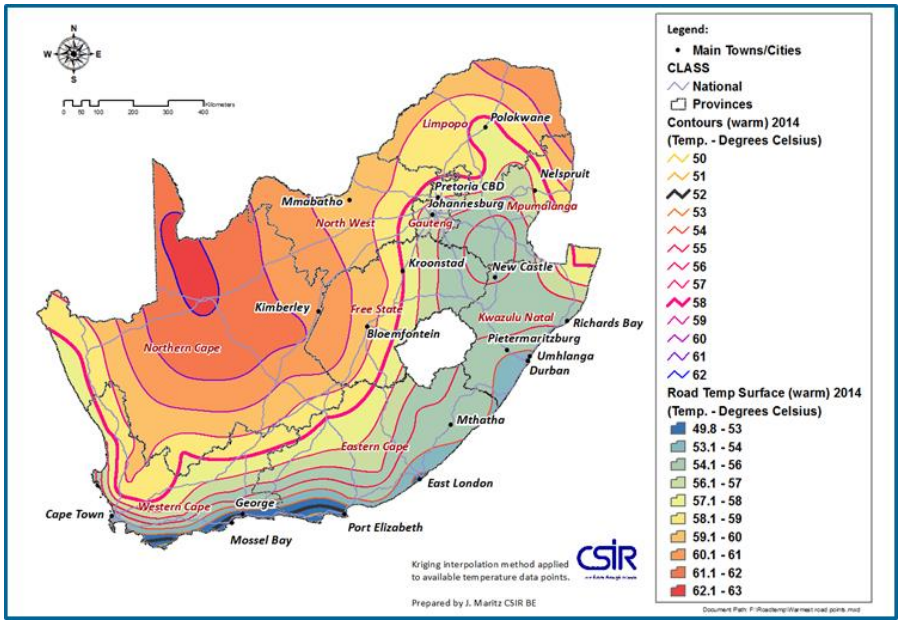

Figure 2: 7-day average maximum asphalt temperatures

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

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

3.1.2 Traffic

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

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

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

Design speeds fall within the following categories:

< 20 km/h
20 – 80 km/h
> 80 km/h

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

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

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

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

The PG binder specifications for South Africa has been published by the South African Bureau of Standards (SABS) as a *technical specification* SATS 3208:2019. Salient compliance limits are given in Table 7. For more details, the reader is referred to the SABS specification.

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

Major advantages of the proposed PG grading include:

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

Table 7: Binder classification and compliance limits

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

3.2 PG binder selection

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

1. Locate the position of the asphalt layer on the map in Figure 2 indicating the 7-day average maximum asphalt design 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
2. Determine the traffic level and average speed and choose the correct grade of binder as indicated in *3.1.2 Traffic*.

3.3 Binder selection for types of asphalt

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

3.3.1 EME

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

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

3.3.2 Sand asphalt

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

3.3.3 Asphalt for lightly trafficked roads in residential areas

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

3.3.4 Porous asphalt mixes

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

3.3.5 Bitumen rubber asphalt

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

3.3.6 Warm mix asphalt

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

3.3.7 Use of Reclaimed Asphalt

Refer to Sabita Manual 36

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

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

3.4 Reclaimed asphalt binder

The effective binder grade after blending with the reclaimed asphalt binder and any rejuvenating agents should be specified for the contract. Practically, this may be determined beforehand by blending virgin binder, binder recovered from the recycled asphalt and rejuvenator in the theoretical proportions and evaluating the blended binder.

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

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

4. Aggregate Selection

As a first principle of the asphalt mix design process the designer should ensure that the aggregate fractions used in preparing the design are representative – within sufficiently narrow bands – of the material in stockpiles or being supplied and which will be utilised during the manufacture of the project mix. Whenever there is a change in source material in terms of e.g. a quarry zone (which may influence unique aggregate characteristics such as density, absorptiveness and surface texture) shape and particle size, the project mix should be redesigned with aggregates representative of the new source.

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 conventional (dense graded) asphalt mixes, the structural and functional performance of an asphalt mix in the pavement layer is significantly 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:[e]

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

4.3 Aggregate sources

4.3.1 Natural aggregates

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

4.3.2 Processed aggregates

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

[e] In certain applications, such as the Bailey method of design, size definitions may be different.

[f] In SMA, which consists of a binary system of aggregate and mortar or mastic, the coarse aggregate is defined in AASHTO R46 as that portion of the total aggregate blend retained on the 5 mm sieve for 14 mm and 20 mm nominal maximum aggregate size. For the 10 mm nominal maximum aggregate size it is that portion of the total aggregate retained on the 2.00 mm sieve.

Manufactured aggregates

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

Slag aggregates

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

Steel slag is a by-product of the steel making process. Utilising steel slag as an aggregate is a means to reduce the large waste stockpiles, as well as to preserve natural resources by not quarrying natural aggregates. The pH is between 8 and 11, and hence it has a strong affinity to bitumen which aids in retaining the binder coating and preventing stripping. This benefits long-term durability, especially in high moisture regions.

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

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

Reclaimed asphalt (RA) aggregate

RA consists of fragments of asphalt that have been removed from the road or sourced from stockpiles of discarded asphalt. Guidelines for sampling of aggregate materials (TMH5 C5) can be followed to sample RA from a stockpile. Segregation is generally a major concern when sampling from RA stockpiles, and care must be taken to avoid it. Processing of RA should be based on recommendations provided in Manual 36: *Use of Reclaimed Asphalt in the Production of Asphalt*.

Note 4.2: When 15 % or more RA is used in asphalt, testing of the RA aggregate and the aged binder is recommended.

4.3.3 Fillers

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

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

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

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

Adequate amounts of filler ensure adequate cohesion, which is a major contributing factor to the provision of resistance to permanent deformation especially in sand-skeleton mixes. Too much filler stiffens the mix, and the mix will be difficult to compact, and too little will result in low cohesion, and the mix may fall apart. Table 8 summarises filler types, characteristics and test methods to determine their properties.

Table 8: Filler types and characteristics

Type of filler/origin	Characteristics	Test method / Criteria
Hydrated lime (active filler)	• Improves adhesion between binder and aggregate • 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/m/ • Voids in compacted filler (BS 812): 0.3 – 0.5%
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	Methylene blue test (SANS 6243): maximum value 5
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	Methylene blue test (SANS 6243): maximum value 5
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 (FBR), i.e. the ratio $\frac{mass\ of\ filler}{mass\ of\ binder}$. It is recommended that the filler-binder ratio of surfacing mixes should not exceed 1.5, particularly for thin-layer mixes that cool more rapidly during paving and compaction. Because of their heat retention, higher filler-binder ratios can be allowed in thick asphalt bases (i.e. a maximum ratio of approximately 1.6). Also, in open-graded mixes such as UTFC, consideration can be given to higher permissible filler-binder ratios: up to 1.7%.

Note 4.4: When active fillers such as cement and hydrated lime are used care should be taken not to increase the viscosity of the hot mastic beyond values that will adversely affect workability during mixing and paving. Where hydrated lime is used the quantity should be limited to 2% by mass of the total aggregate.

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

4.4 Aggregate shape

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

4.5 Aggregate grading

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

Table 9: Changes in sieve sizes from TMH1 to SANS

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

4.5.1 Coarse aggregate grading classes

While, in general, the nominal single sized coarse aggregate fractions of the blend should comply with the provisions of Table A9.1.5-3 of the COTO specification, the gradings of nominal single sized coarse aggregate fractions for *stone-skeleton mixture types* should meet the requirements of grading Class 1, of the abovementioned table. These requirements for a Class 1 grading class are given in Table 10 for a range of NMPS and are particularly pertinent to:

- Stone mastic asphalt (SMA)
- Ultra-thin friction courses (UTFC)
- Porous asphalt mixes

Table 10: Class 1 grading limits for nominal single size coarse aggregates

Sieve size (mm)	Nominal maximum particle size (NMPS) (mm)					
	28.0	20.0	14.0	10.0	7.1	5.0
	Percentage passing sieve size by mass					
37.5	100					
28.0	85 - 100	100				
20.0	0 - 20	85 - 100	100			
14.0	0 - 5	0 - 20	85 - 100	100		
10.0		0 - 5	0 - 20	85 - 100	100	
7.1			0 - 5	0 - 20	85 - 100	100
5.0				0 - 5	0 - 20	85 - 100

4.5.2 Fine aggregate grading

The gradings for fine aggregate material should be such that the maximum size, size distribution and the material uniformity is appropriate for the mix type. Materials with a maximum size of 5mm or even 7mm may be suitable for use with sand-skeleton mixtures. However, due to the relatively high proportion of a fine aggregate in these mix types, assuring uniformity in quality would require special attention. While the proportion of fine aggregate in stone-skeleton mixtures is significantly less, the grading limits have to meet specific requirements so as to attain the critical properties of the composite structure.

It is also recommended that the Fineness Modulus of both crushed and natural fine aggregates should not deviate by more than 0.2 from the value reflected in the approved design.

In some cases it may be expedient to permit the use of natural fines – i.e. material not obtained from crushed rock – provided, of course, that the specified mix properties are met. It is good practice to limit the proportion of such natural fines to 7% by mass of the fine aggregate, for mixes other than sand-skeleton, gap– and semi-gap graded mixtures. In addition, the liquid limit should not exceed 25% and the PI should not exceed 4. Natural fines should be introduced into the mixing plant by means of a separate plant cold-feed bin.

4.6 Mix grading requirements

4.6.1 Grading control points

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

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

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

Table 11 provides grading control points for four nominal maximum particles sizes of aggregates typically used for production of ***sand-skeleton*** **(continuously graded)** asphalt mixes in South Africa. The control points for 20 mm NMPS are plotted on a 0.45 power chart in for illustration purposes in Figure 3.

Table 11: Aggregate grading control points

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

Note 4.7: The control points given in Table 11 ***should be used as a guideline*** and a starting point preceding a specific method of aggregate proportioning, such as the Bailey method[g], as set out in Appendix A. In cases where the bulk density of the different aggregate fractions to be used in the mixture vary by more than 200 kg/m^3 the trial blend gradations should be based on volumetric percentage.

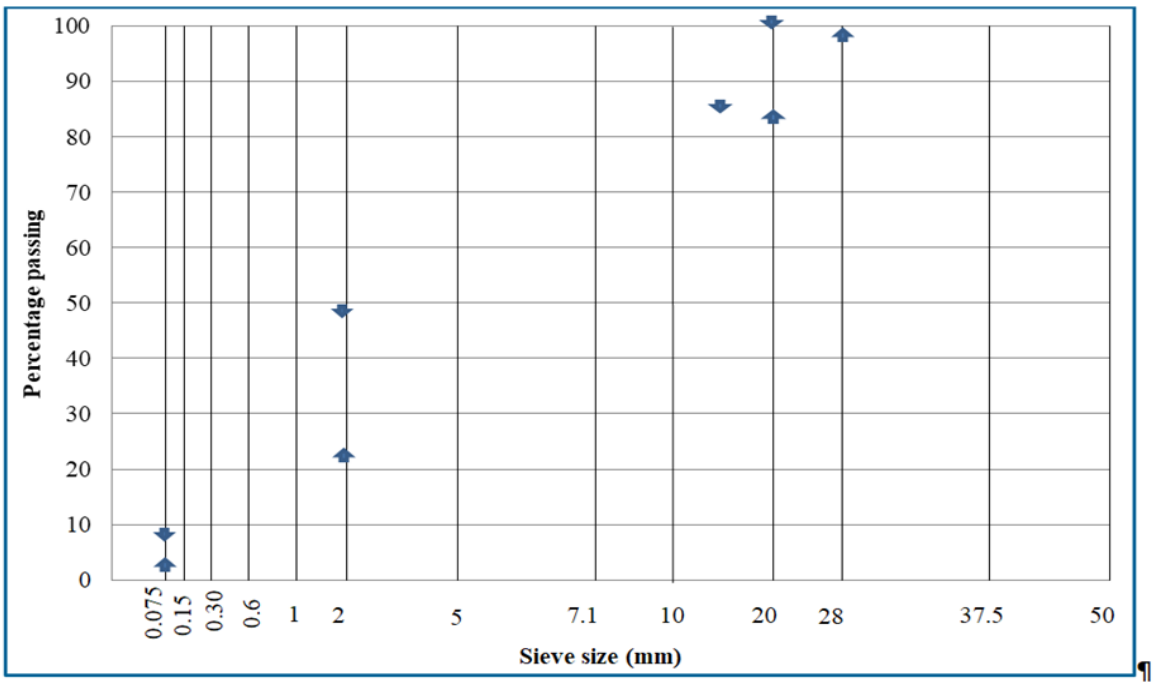

Figure 3: Grading control points plotted on 0.45 power chart for NMPS = 20 mm

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

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

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

An overview of the method is provided in Appendix A.

4.7 General requirements and specifications for aggregates

The following requirements are generally applicable to aggregates for asphalt:

- Coarse and fine aggregates obtained from crushing or natural sources should be clean and free from decomposed materials, vegetable matter and other deleterious substances;
- The aggregate blend may contain natural fines not obtained from the parent rock being crushed, subject to limitations of the proportion of such materials based on mix type and experience with the materials;
- The coarse aggregate is, in most cases, crushed rock. Certain types of crushed blast-furnace slag may also be used, provided they satisfy the strength requirements and are not excessively 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 12.

4.8 Preparation and selection of aggregate grading

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

- Source samples of raw aggregate materials from stockpiles at asphalt plants in accordance with Sabita Manual 35. Each stockpile usually contains a given size of an aggregate fraction. A minimum of three fractions are used to generate a combined grading for the mix. These aggregates must be clean and free from decomposed materials, vegetable matter and other deleterious substances.
- Oven dry aggregates to constant mass at approximately 105°C. Samples for sieve analysis are reduced by (riffling / quartering). Ensure homogeneity of samples by mixing together, bags of similar aggregate sizes.
- Conduct wet sieve analysis test (SANS 3001-AG1) on randomly selected bags of samples to check if aggregates are adequately riffled. Determine the bulk and apparent densities for each coarse and fine aggregate fraction as per SANS 3001-AG20 and 3001-AG21, respectively. Also determine the bulk density of the mineral fillers as per BS 812 procedures.

Determine properties of individual aggregate fractions. The recommended test methods and criteria are presented in Table 12.

- Combine the individual aggregate fractions into trial blends of a single grading by using a basic formula presented in Equation 4.1. Blends can be obtained by trial and error using Excel Solver or any commercially available software that does aggregate blending.
  P = Aa + Bb + Cc, … Eq. 4.1)

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

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

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

- Prepare a minimum of three trial aggregate blends; plot the grading of each trial blend on a 0.45-power chart, and, ***for sand-skeleton mix types***, compare the gradings of the trial blends with the guidelines provided in Table 11 (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 12: Recommended tests and criteria for aggregate selection

Property	Test	Standard	Criteria
Hardness / Toughness	Fines aggregate crushing test: 10% FACT	SANS 3001-AG10	Sand-skeleton mixes: ≥ 160 kN Stone-skeleton mixes: ≥ 210 kN
	Aggregate crushing value (ACV)	SANS 3001-AG10	Sand-skeleton mixes ≤ 25 Stone-skeleton mixes ≤ 21 Rolled in chippings ≤ 21
Soundness	Magnesium sulphate soundness	SANS 5839 SANS 3001-AG12	12% to 20% is normally acceptable. Some specifications require ≤ 12% loss after 5 cycles
Durability	Methylene blue adsorption indicator	SANS 6243	High quality filler: ≤ 5 > 5: additional testing and analysis needed
Particle shape and texture	Flakiness index	SANS 3001- AG4	20 mm and 14 mm aggregate: ≤ 25[h] 10 mm and 7.1 mm aggregate: ≤ 30 Rolled in chippings ≤ 20
	Polished stone value (PSV)	SANS 3001–AG11	Minimum 50[i]
	Fractured faces	ASTM 5821	Sand-skeleton mixes: at least 50% of all particles should have three fractured faces Stone-skeleton mixes & rolled in chippings: at least 95% of all particles should have three fractured faces
Water absorption	Coarse aggregate (> 5mm)	SANS 3001-AG20	≤ 1% by mass
	Fine aggregate (< 5mm)	SANS 3001- AG21	≤ 1.5% by mass
Binder absorption	Coarse and fine aggregate	SANS 3001-AS11	≤ 0.5% by mass
Cleanliness	Sand equivalency test	SANS 3001-AG5	≥ 50 total fines fraction
	Clay lumps and friable particles	ASTM C142–97	≤ 1%

[h] For certain mix types, especially stone-skeleton types, e.g. UTFC and SMA, more stringent requirements are usually specified

[i] Consideration can be given to adopting a limiting value of 45, with due regard to material availability, traffic, road geometry and climate

4.9 Surface area of aggregate

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

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

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

This calculation is based on a bulk density of the -5 mm fraction of the total aggregate of 2 650 kg/m^3. Consequently, in cases where the bulk density of this aggregate fraction has a different value, a correction is required as follows: $SA_N = SA \times \frac{2\,650}{BD_5}$ (Eq. 4.3) where: SA_N = Normalised specific surface area BD_5 = Bulk density of the aggregate fraction passing the 5 mm sieve, kg/m^3

5. Mix design

5.1 Introduction

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

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

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

5.2 Asphalt mix properties

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

5.2.1 Workability

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

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

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

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

5.2.2 Durability

Durability of asphalt mix is its ability to resist:

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

Durability of mixes can be improved by using:

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

5.2.3 Stiffness

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

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

5.2.4 Resistance to permanent deformation (Rutting)

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

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

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

5.2.5 Resistance to fatigue cracking

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

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

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

5.2.6 Permeability

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

Factors that reduce permeability are:

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

Since water permeability tests are often problematic in that high variability can be experienced given the complexity of flow paths for a particular testing location, air permeability is prescribed to quantify permeability during the design phase with limits based on experience. Water permeability can be used on site as a quality assurance tool.

5.2.7 Thermal fracture

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

5.3 Composition of asphalt

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

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

5.4 Mass and volumetric concepts and definitions

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

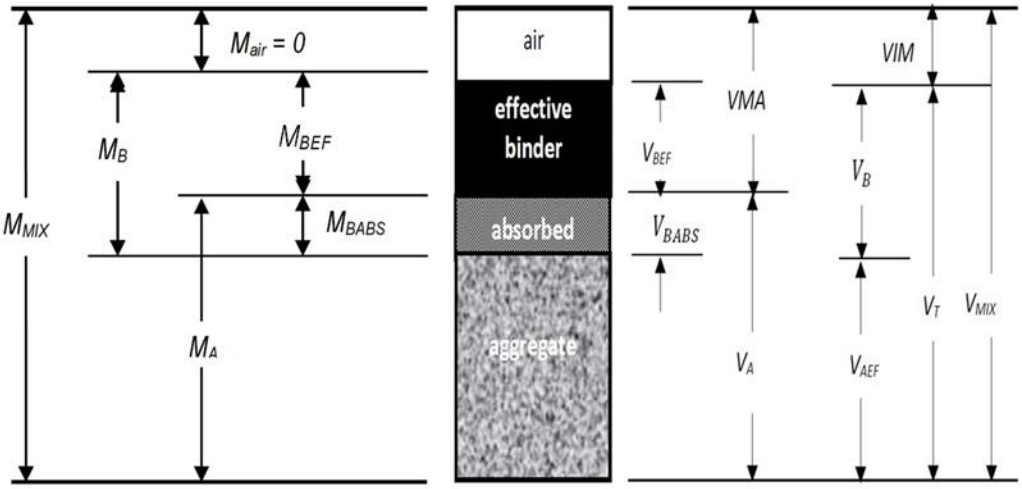

Figure 4: Volumetric parameters of a compacted asphalt specimen

Both the mass and volume parameters in the above illustration are defined in Table 13.

Table 13: Mass and volume parameter definitions

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

Table 14 lists the definitions of the density parameters and associated test standards used routinely in the design of asphalt mixes. The definitions of density parameters for aggregate are schematically illustrated in Figure 5 (on page 39).

Table 14: Density parameters used in volumetric analysis

Parameter	Symbol	Definition	Method
Bulk density of aggregate	BD_A	Mass of the aggregate particles divided by the volume of the aggregate particles including the impermeable (internal), and permeable (surface) pores, but excluding the inter-particle voids, expressed in kilograms per cubic metre (kg/m³)	SANS 3001-AG20 (> 5 mm) SANS 3001-AG21 (< 5 mm)
Apparent density of aggregate	AD_A	Mass of the aggregate particles divided by the volume of the aggregate particles including impermeable (internal) voids but excluding any pores or capillaries that become filled with water, after soaking for15 – 19-hours duration, and inter-particle voids, expressed in kilograms per cubic metre (kg/m³)	SANS 3001-AG20 (> 5 mm) SANS 3001-AG21 (< 5 mm)
Effective density of aggregate	BD_{AEF}	Mass of the aggregate particles divided by the volume of the aggregate particles excluding the volume of pores that absorb binder and inter-particle voids, expressed in kilograms per cubic metre (kg/m³)	
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)	ASTM D70
Bulk density of mix[j]	BD_{MIX}	Mass per unit volume, including the air voids, of a bituminous mixture at a known test temperature, expressed in kilograms per cubic metre (kg/m³)	Sabita Manual 39: ASP8
Maximum voidless density of the mix (Rice method)	MVD	Mass per unit volume of a voidless bituminous mixture at a known test temperature, expressed in kilograms per cubic metre (kg/m³)	Sabita Manual 39: ASP9
Coarse aggregate loose unit weight	LUW	The loose unit weight of an aggregate is the amount of aggregate that fills a unit volume without any compactive effort applied.	AASHTO T-19
Coarse aggregate rodded unit weight	RUW	The rodded unit weight of aggregate is the amount of aggregate that fills a unit volume with compactive effort applied	AASHTO T-19

Note 5.1: For the purpose of calculations, the bulk density of penetration grade binder may be taken as 1 020 kg/m³. However, the density of bitumen may vary significantly from this figure and the designer should approach the supplier to ensure that the value adopted for design purposes is relevant to the bitumen that will be used in the project. Where modified binders are used the value of the bulk density of the binder should be obtained from the supplier.

5.5 Determination of mix design parameters

5.5.1 Bulk density of the mix

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

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

In the AASHTO T 331 standard the bulk *specific gravity* (*Gmb*) of the specimen is determined. The bulk density of the specimen BD_{MIX}, in kilogram per cubic metre (kg/m^3), is calculated as follows:

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

where:

Gmb is the bulk specific gravity of the specimen;
ρ_W = density of water at 25°C, (997.1 kg/m^3).

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

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

or

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

where:

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

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

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

where:

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

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

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

where:

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

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

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

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

5.5.2 Maximum Voidless Density of the Mix

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

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

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

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

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

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

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

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

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

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

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

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

5.5.3 Notes on density measurements

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

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

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

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

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

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

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

These volumetric concepts (denominators in density expressions) related to aggregates are illustrated conceptually in Figure 5. The volumes associated with the determination of densities listed are indicated with blue outlines.

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

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

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

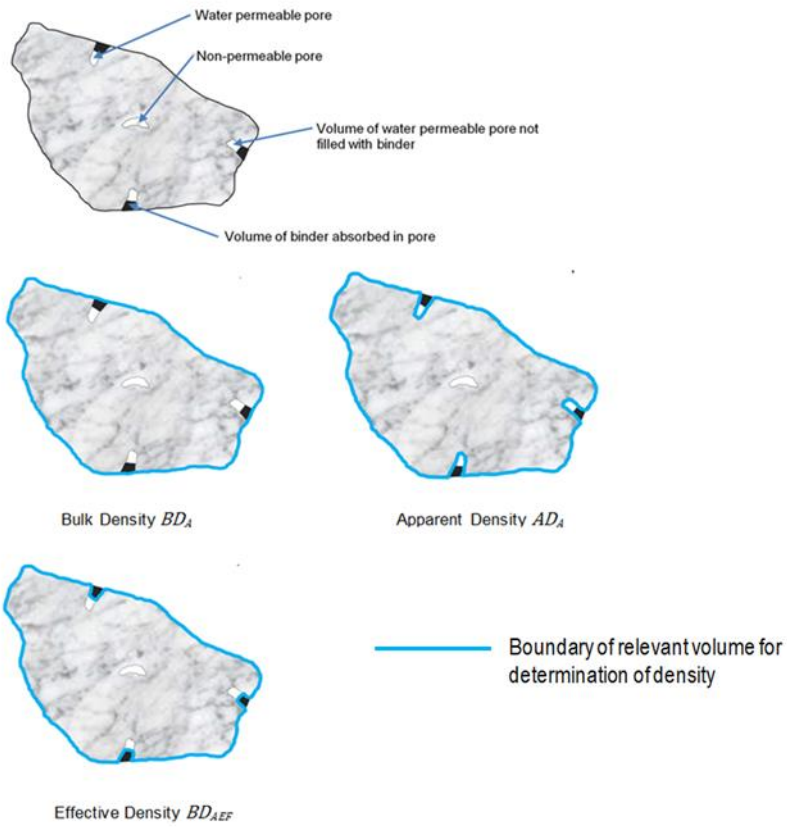

Figure 5: Conceptual illustration of the volumes associated with the determination of Bulk, Apparent & Effective Density of aggregates

5.5.4 Additional volumetric calculations

Other compositional parameters and their definitions used routinely in the design of asphalt are listed in Table 15.

Table 15: Compositional parameters used in volumetric analysis

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

5.6 Mix design levels

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

In terms of the traffic classification given in Table 1, the mix design levels being adopted in terms of the desired risk profile are shown in Table 16.

Table 16: Mix design levels for traffic volumes over service life of the pavement

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

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

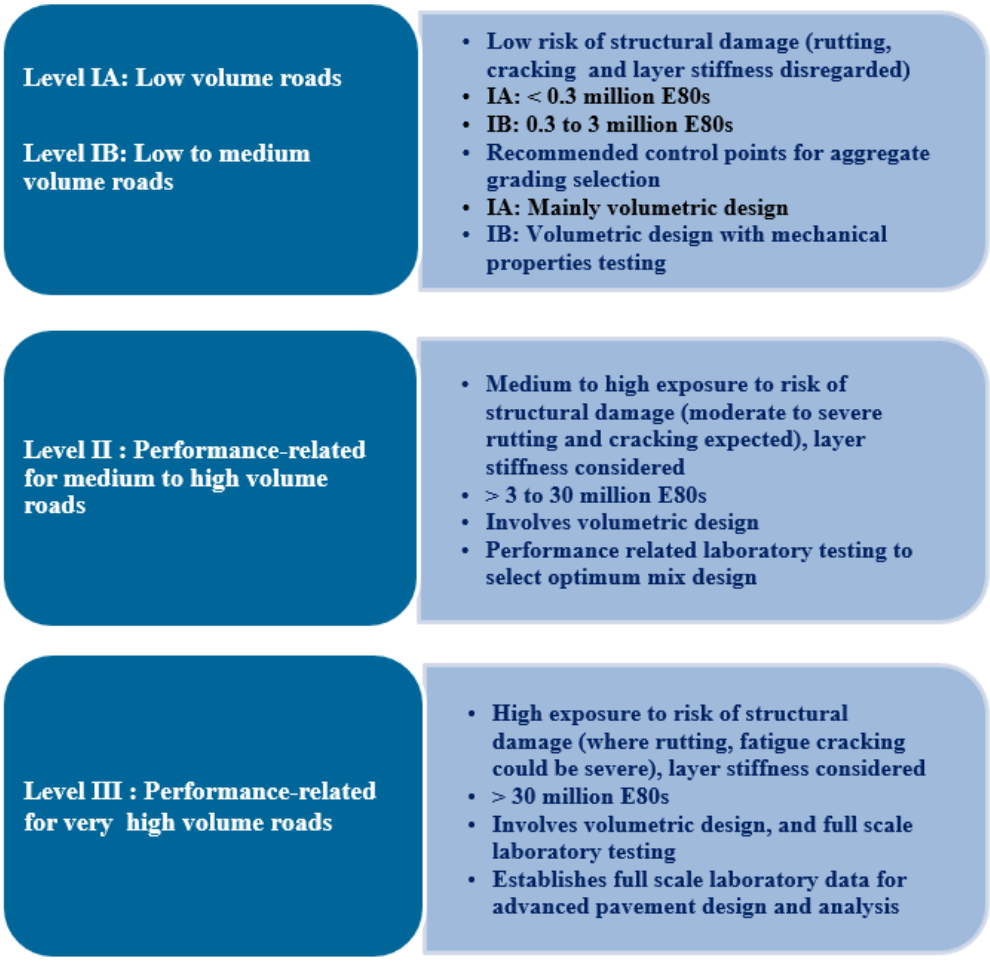

Figure 6: Mix design levels

5.6.1 Level I mix design process

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

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.

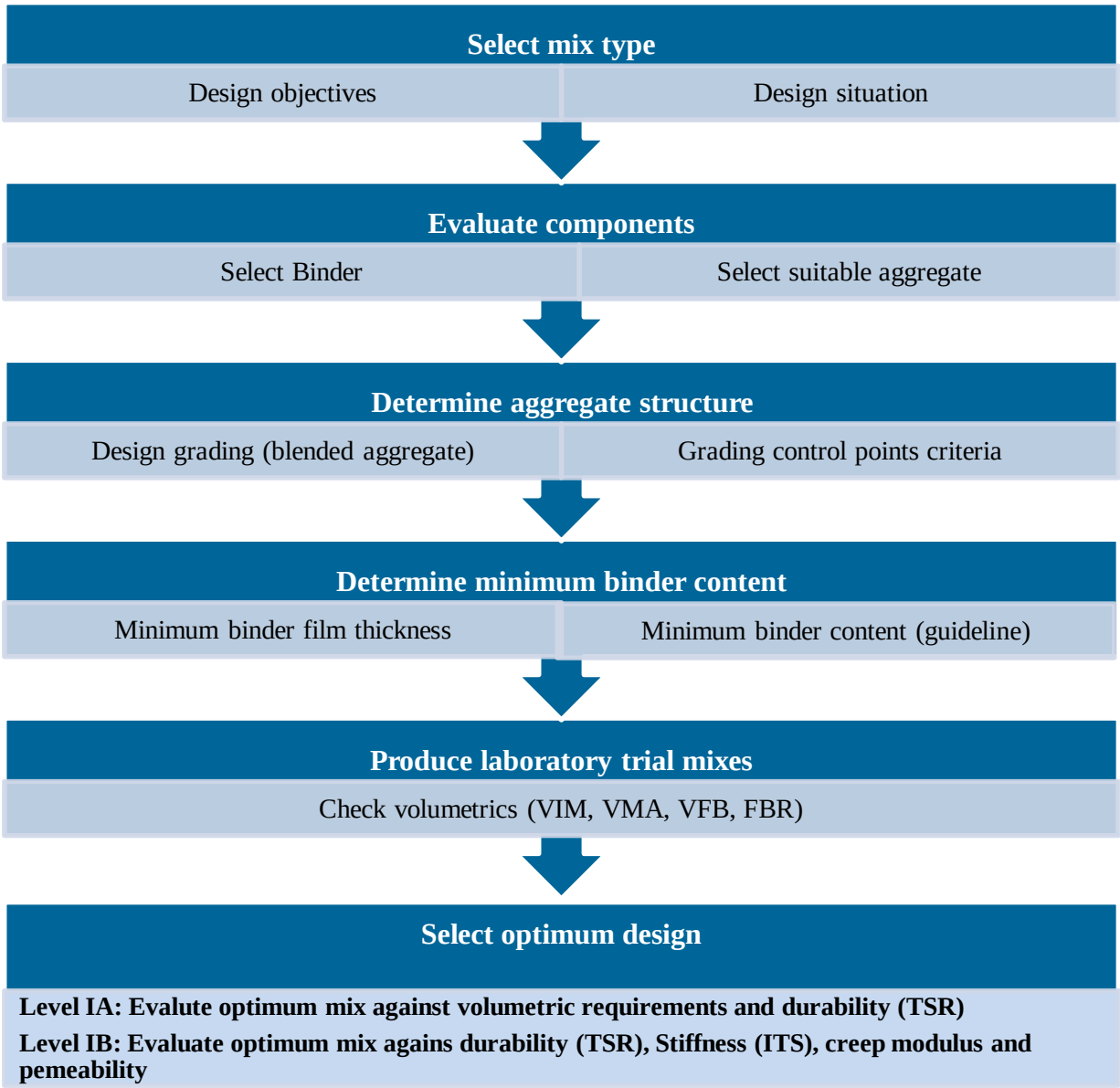

Figure 7: Level I design process

(4) Develop three trial aggregate blends (gradings) from the selected aggregate fractions. The design aggregate composition is established by:

(a) Determine minimum binder content for each trial blend using the minimum requirements for binder film thickness, specific surface area and density of the –5 mm fraction of the aggregate blend. In calculating the binder film thickness, the designer should note that the volume of binder used is the ***effective binder***, i.e. the volume of binder NOT absorbed by the aggregate. Eq. 5.1 yields the film thickness, F, in µm (micron):

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

where:

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

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

BD_B is the density of the binder at 25°C

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

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

Note 5.2 The determination of the specific surface area according to Eq. 4.2 above yields a ***theoretical value*** and, given the wide variety of aggregate shapes and textures, is not a precise computation of the actual area. Its value lies in being a ***consistent comparative parameter.***

Note 5.3 In determining the minimum binder content, the requirements in respect of the filler / binder ratio should be given due consideration.

Note 5.4 Use of the binder film thickness, F, is recommended practice for asphalt in general use, i.e. sand and stone-skeleton mix types. When designing EME, the determination of minimum binder content to satisfy a *Richness Modulus* requirement is the correct practice, as per Sabita Manual 33. Additionally the binder content of SMA should be designed in accordance with Appendix B.

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

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

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

i. Select four trial binder contents based on; (1) minimum binder content, (2) minimum binder content + 0.5%, (3) minimum binder content + 1.0%, and (4) minimum binder content + 1.5% by mass of total mix.
ii. Determine filler-binder ratio
iii. Prepare[I] three duplicate mixes for each trial binder content. Each trial binder content should be mixed with the same design aggregate composition. Also prepare two loose asphalt samples specimens to determine the maximum void-less density (MVD) as per SANS 3001-AS11.
iv. Specimens should then be short-term aged in accordance with Sabita Manual 39.
v. Samples should be mixed and compacted at the appropriate mixing and compaction temperatures based on the selected binder type or grade. Mixing temperature is the range of temperatures that yields a binder viscosity (rotational) of approximately 0.17 ± 0.02 Pa.s, whereas the compaction temperature is obtained at viscosity of 0.28 ± 0.03 Pa.s. The appropriate temperatures for mixing and compaction should be based on temperature / viscosity information provided by the supplier of the bituminous binder in use.

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

vi. Compact the trial specimen (102 mm in diameter by 64 mm in height) for each trial binder content. The standard Marshall method for making asphalt briquettes (as contained in SANS 3001-AS1) should be followed, except that the recommended number of blows to compact the specimens is given in Table 17.

vii. Determine the bulk density (BD) of the compacted samples as set out in section 5.5.1 *Bulk density of the mix*

viii. Use the BD and MVD results (average values for each trial binder content) to obtain the following volumetric parameters of the mix:
- Voids in the total mix (VIM).
- Voids in the mineral aggregate (VMA).
- Voids filled with binder (VFB).

ix. The criteria for Voids in the Mix presented in Table 18 must be met for the mix.

Table 17: Compaction requirements

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

Note 5.5 The values of VMA and VFB given in Table 19 and Table 20, respectively, are given to guide the designer in determining the optimum composition of aggregate fractions of sand skeleton mixes so as to reasonably ensure that critical parameters such as binder film thickness, voids and durability requirements will be met. ***The values of VMA and VFB should not be considered as a requirement for specification purposes.***
These values apply to ***continuously graded, sand-skeleton mixes*** only. The volumetric requirements for stone-skeleton mixes should be determined using appropriate documentation.

Table 18: Voids requirements

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

Table 19: Minimum percent VMA

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

Table 20: Percent VFB

Minimum	Maximum
70	80

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

x. Determine optimum binder content based on the combined results of Marshall stability and flow (SANS 3001-AS1), density and void analyses as follow:
   - Draw graphs of the following six relationships:
     - Bulk density versus binder content.
     - Marshall Stability versus binder content.
     - Marshall Flow versus binder content.
     - Air voids versus binder content.
     - VMA versus binder content.
     - VFB versus binder content.
   - From the graph of air voids versus binder content, determine the binder content at 4%.
   - From the graph of bulk density versus binder content, determine the binder content at maximum (peak) bulk density.
   - From the graph of stability versus binder content, determine the binder content at maximum (peak) stability.
   - The three binder contents selected at 4% air voids, maximum BD and stability are averaged to determine the optimum binder content.

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

Note 5.6: The sole purpose of the Marshall method in this design procedure is to determine the optimum binder content for Level IA design.

Note 5.7: Where required the durability of the optimum mix can be assessed (as in Level IB) by conducting the Modified Lottman testing (ASTM D4867M) on the mix. See Table 23 below.

<u>For Level IB mix design</u>

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

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

ii. For each trial blend, compact the three duplicate specimens (150 mm in diameter by 115 mm in height) in a Superpave gyratory compactor following the test procedures contained in AASHTO T 312. Also, prepare two loose asphalt samples for each trial mix for determination of the maximum void-less density (MVD) of the mix using SANS 3001-AS11. Compact specimens immediately after completion of short-term oven conditioning to N_{design} in accordance with Table 21.

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

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

Table 21: Compaction requirements for Level IB

Design traffic (E80s)	N_{design}
0.3 – 3 million	75

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

[n] See note 5.2

i. Use the volumetric data to generate graphs of VIM, VMA and VFB versus binder contents. The design (optimum) binder content is established at 4 percent air voids (on the VIM versus binder content graph). The VMA and VFB are checked at the design binder content to verify that they meet the criteria presented in Table 19 and Table 22. Again, these values are given as guidance as set out in Note 5.5.

ii. The durability of the optimum mix design is assessed by conducting the Modified Lottman testing (ASTM D4867M) on the mix. Prepare short-term aged loose samples and compact the specimens to in-place voids (typically, 7% ± 0.5% for continuously graded mixes). A reasonable rule of thumb is that in-place voids are approximately equal to design voids +3%. Calculate the tensile strength ratio, and check results against the criteria presented in Table 23.

A summary of the requirements and criteria to attain the optimum design for Level IB are given in Table 24.

iii. 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 22: Recommended percent VFB

Minimum	Maximum
65	75

(c) Notes on interrelationships of volumetric parameters

Note 5.8: 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 aggregate particles with a rougher surface texture.

Note 5.9: The effect of grading on VMA is somewhat complex; however denser gradings generally lead to a decrease in VMA. Also larger aggregates (NMPS) reduce VMA. Low VMA is very sensitive to slight changes in binder content. Generally, economising the binder content by lowering VMA is counter-productive and should be avoided.

Note 5.10: VFB restricts the allowable air void content for mixes which are near the minimum VMA criteria. Mixes designed for lower traffic volumes may not pass the VFB requirement with a relatively high percent air voids in the field even though the air void range requirement is met. Also, in mixes requiring high voids content and relatively thick binder films, e.g. open graded mixes and UTFC's the maximum limit of 75% will be exceeded. Meeting VFB requirements avoids less durable mixes resulting from thin films of binder on the aggregate particles.

Note 5.11: 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 aggregates with a rougher surface texture.

Table 23: Moisture resistance criteria (Minimum TSR)

Layer type	
Base	Wearing course
0.70	0.80

Note 5.12: 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 IB

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
Permeability	Air permeability	Sabita Manual 39: ASP5	$\leq 1 \times 10^{-8}$ cm^2

Note 5.13: Stone-skeleton mixes and mixes manufactured with some polymer modified or bitumen-rubber binders may have low dynamic creep and ITS values and still exhibit good resistance to rutting. This test may therefore not be applicable for such mixes.

5.6.2 Level II and Level III design process

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

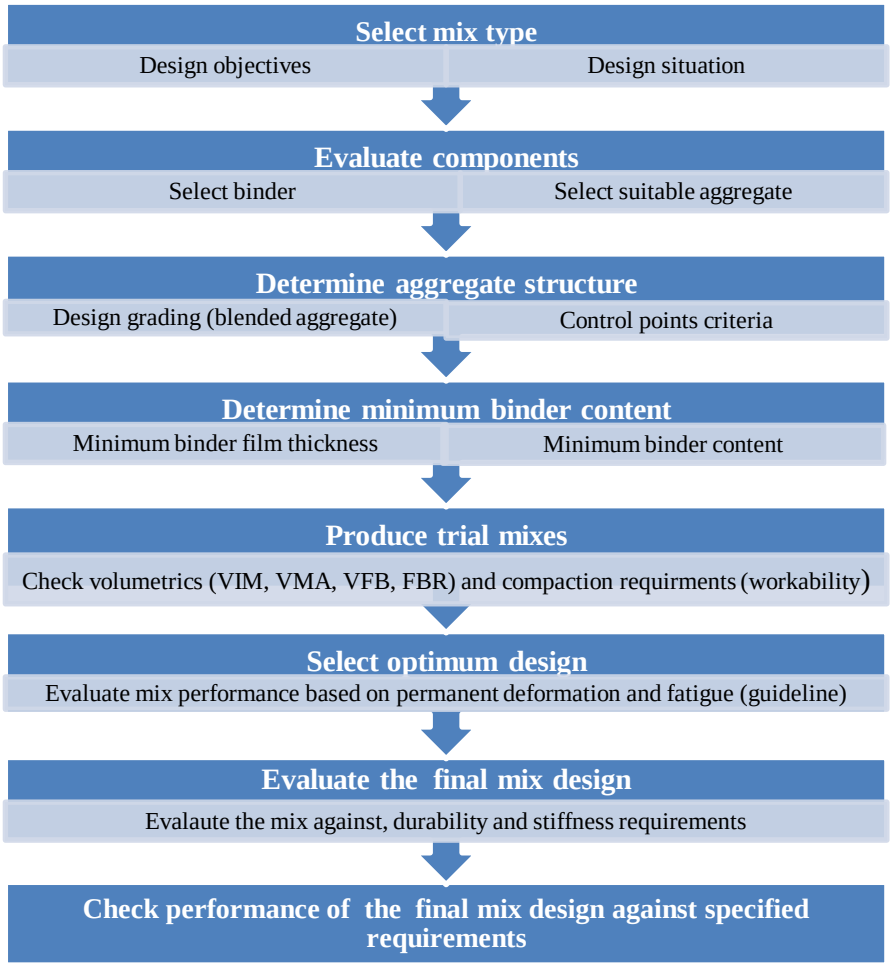

Figure 8: Level II and Level III design process

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

(1) Select optimum mix. The selection of optimum design at these levels involves the same sample preparation and determination of volumetrics as described for Level I except that only the Superpave gyratory compactor (AASHTO T 312) test procedure is used. VMA is checked at the design binder content to verify that the recommended requirements in Table 19 are met. Compaction and VFB recommendations for Level II and Level III as presented in Table 25 and Table 26 are different.

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

Table 25: Laboratory compaction requirements for Levels II & III

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

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

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

Note 5.14: The workability criteria serve as a guide only and clearly should be considered in conjunction with a number of factors such as shape and surface texture of available aggregate and mix type (i.e. stone– or sand-skeleton).

(3) Evaluate durability of the mix by using the Modified Lottman test procedures (ASTM D4867M), and check results against the criteria set in Table 23.

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

Note 5.15: At Level II design, dynamic modulus test is conducted at frequency sweeps of 0.1, 0.5, 1, 5, 10, and 25 Hz at one test temperature of 20°C. At Level III design, a full factorial test of dynamic modulus is conducted at the five frequencies above and at five temperatures (-5, 5, 20, 40 and 55°C).

(5) Select the optimum mix design based on performance

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

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

Note 5.16: The Hamburg Wheel Tracking Test (HWTT) indicates susceptibility to premature failing of asphalt mixtures due to weak aggregate structure, inadequate binder stiffness, moisture damage, and inadequate adhesion between aggregate and binder. HWTT results are influenced by aggregate quality, binder stiffness, duration of short-term ageing, binder source, anti-stripping treatments and compaction temperature. The test outputs include post-compaction consolidation, usually assessed at 1 000-wheel passes, creep slope, stripping slope, and stripping inflection point, as illustrated in Figure 9.

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

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

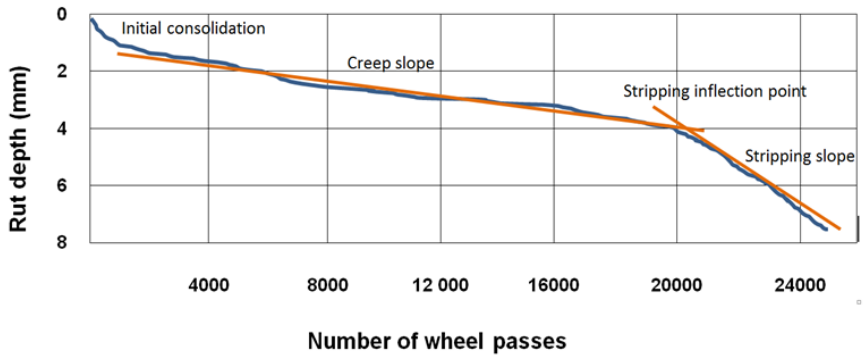

Figure 9: Definition of the Hamburg Wheel Tracking Test results

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

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

These compliance criteria are set out in Table 27.

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

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

It is suggested that, in cases where the rut depth developed after the prescribed minimum number of passes exceeds 6 mm, further investigation be carried out , e.g. MMLS testing (see note 5.19).

Note 5.17: HWTT data validation/confirmation is to be performed on plant trial material and included for the determined working mix design selected for the project. Care should be taken to note the age condition of the mixture tested to allow informed analysis/comparison of results to be done between laboratory design conditions and plant manufactured conditions.

Table 27: Criteria for permanent deformation

Design Traffic [million E80]	Passes to 6 mm rut [Temperature zone]	SIP [min]	Mix Design Level
>3 - 10	16000 [58]	10 000	II
	20 000 [64, 70]		
> 10	20 000 [58, 64, 70]		II, III

- ***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 test as described in AASHTO T 321.

  i. Prepare slabs from compacted mix and cut the beams (380 mm long by 63 mm wide by 50 mm high) to conduct the fatigue test. A minimum of 9 specimens are prepared and tested at the design voids and design binder content for Level II design and a minimum of 27 specimens for Level III design, based on three repeat tests per test condition.

  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 a fatigue curve for the mix (three repeats).

  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 (three repeats).

  iv. The selected strain levels will depend largely on the type of binder in the mix. For conventional binders initial peak-to-peak strain levels within the range of 250 to 750 με may be appropriate; mixes with highly modified binders may require initial strain levels as high as 2 000 με. In accordance with AASHTO 321 strain levels should be such that at least 10 000 load cycles are applied. Accordingly it is considered that a practical upper limit of duration for testing a specimen should be two days (48 hours).

  v. Fatigue failure is defined as the load cycle (n) at which the product of the specimen stiffness (S) and the loading cycles, i.e. ($S \times n$) is a maximum with respect to n. (See Figure 10)

  vi. Mix-specific fatigue models to estimate fatigue performance of an asphalt pavement layer can be derived in conjunction with the dynamic modulus of the mix as part of the pavement design process.

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

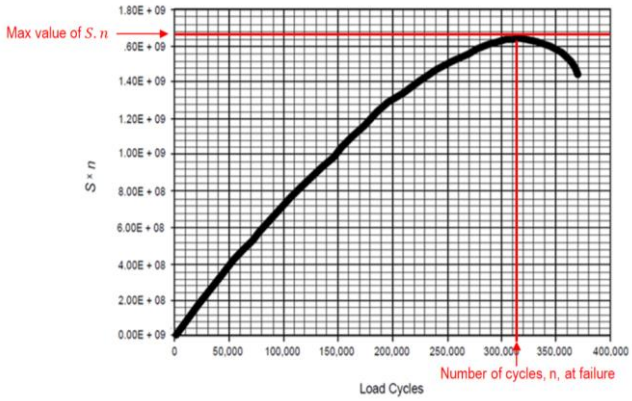

Figure 10: S x n versus Load Cycles

Note 5.18: All specimens compacted for the three mix design levels must be short-term aged in accordance with Sabita Manual 39.

Note 5.19: Although not a specific requirement, the use of the MMLS3 as per testing protocol SANS 3001-PD1:2014 is recommended should additional investigation and site validation testing be indicated. (See note 5.17). Table 28 lists test properties testing conditions, and the number of compacted specimens required to conduct laboratory test for Level II and Level III designs.

Table 28: Summary of performance-related tests

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

5.7 Design of special mixes

Detailed guidelines and production methodologies with recommendations and criteria are available for the following special mixes to supplement or even override some of the procedures set out in this design manual:

5.7.1 Bitumen-rubber asphalt

Reference document: Sabita Manual 19: *Guidelines for the design, manufacture and construction of bitumen-rubber asphalt wearing courses*

5.7.2 Porous asphalt

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

5.7.3 Mixes for light traffic in residential areas

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

5.7.4 Warm mix asphalt

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

5.7.5 EME asphalt

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

5.7.6 Mixes with reclaimed asphalt

Reference document: Manual 36*: 2016: Use of reclaimed asphalt in the production of asphalt*
The following limitations on the percentages of reclaimed asphalt in a mix are recommended:

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

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

5.7.7 Sand asphalt

Reference document: Sabita Manual 18: Appropriate Standards for the Use of Sand Asphalt

5.7.8 Stone mastic asphalt (SMA)

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

6. Link with asphalt pavement design

6.1 South Africa pavement design method

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

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

6.2 Asphalt pavement layer considerations

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

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

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

The following models for asphalt materials are included in the revised SAPDM [52]:

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

6.3 Resilient response of asphalt

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

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

- Binder ageing model;
- Dynamic modulus models;

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

6.3.1 Basic level

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

6.3.2 Intermediate level

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

6.6.3 Advanced level

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

6.6.4 Predicting dynamic modulus from laboratory data

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

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

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

where:

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

f_r .= reduced frequency [Hz]

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

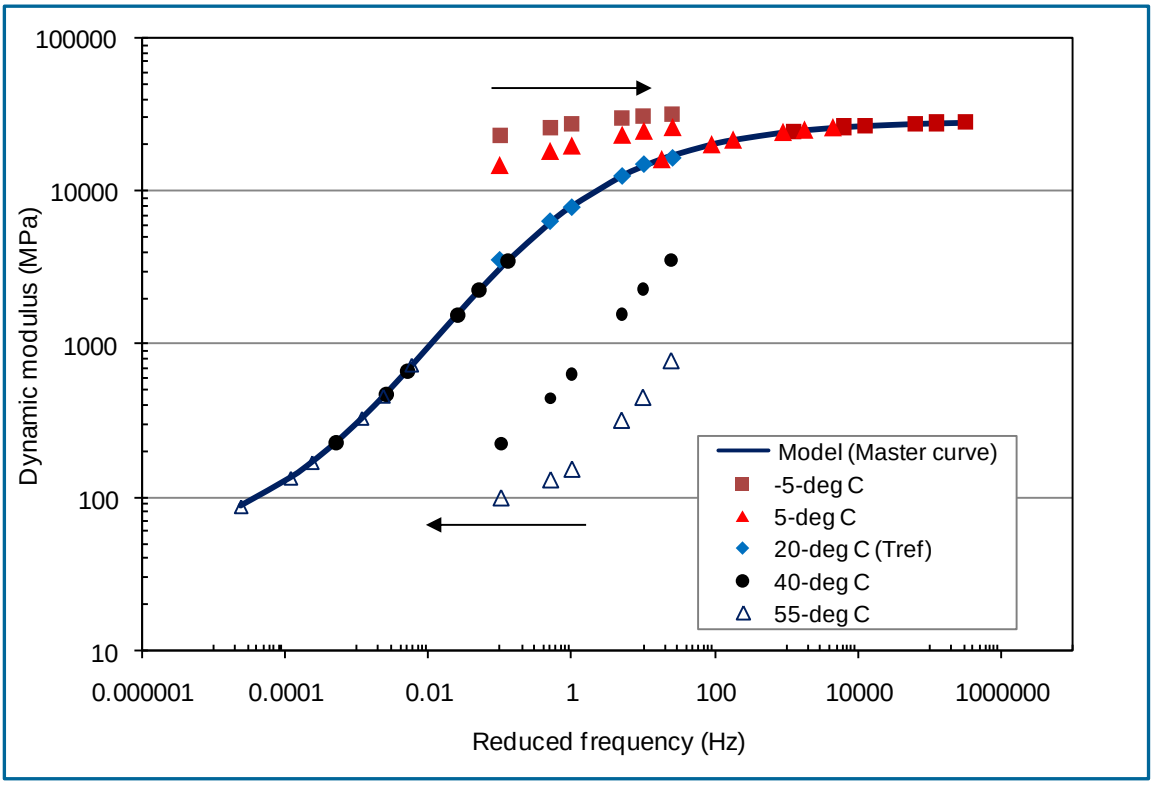

Figure 11 : Typical master curve for dynamic modulus (Anochie-Boateng et al. 2011)

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

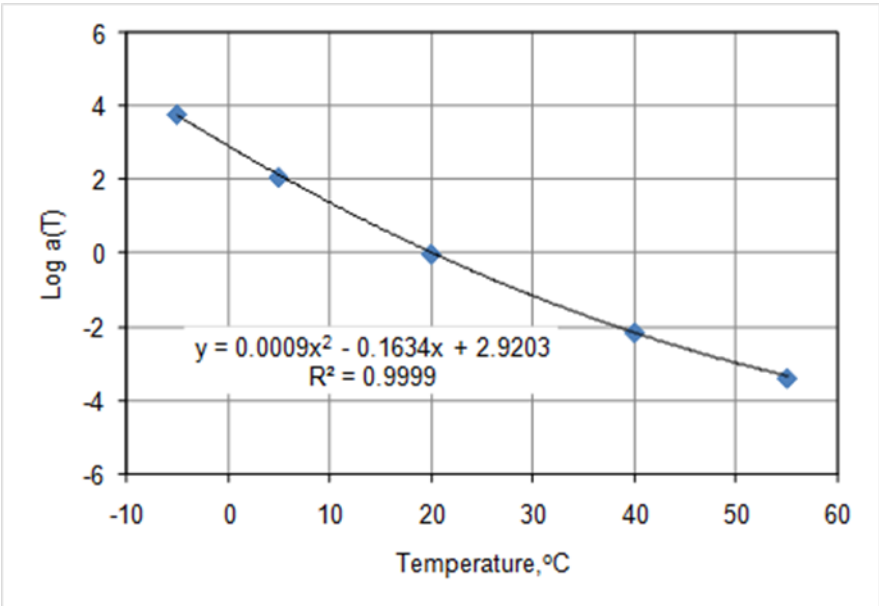

Figure 12: Temperature shift factors for master curve (reference temperature – 20°C)

6.4 Damage modelling

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

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

6.5 Long life pavement

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

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

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

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Appendix A

Overview of the Bailey Method for Determining Aggregate Composition

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

A.1 Aggregate grading

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

A.2 Definitions

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

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

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

A.3 Unit weight of aggregates

Unit weight is the traditional terminology used to describe the property determined in the Bailey method, which is weight per unit volume (mass per unit volume or density). Table 29 shows unit weights and test methods used in the Bailey concepts. Table 30 presents recommended chosen unit weights of mix types, Table 31 indicates recommended voids ranges of loose and rodded unit weights, whereas the characteristics of the various mix types are presented in Table 32.

Table 29: 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% – 49% $V_{LUW} = 100 \times \left[\frac{BD_A - LUW}{BD_A} \right]$

Rodded unit weight (RUW)	• Requires compactive effort
	  ○ Three layers
	  ○ Rodded 25 times each
	• Increased particle-to-particle contact
	• Determine RUW (kg/m³)
	• Determine volume of voids
		AASHTO T19	V_{RUW} [2]: 37% – 43%
			$V_{RUW} = 100 \times \left[\frac{BD_A - RUW}{BD_A} \right]$

Chosen unit weight (CUW) (Table 30)	• 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 30

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

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

Table 30: Recommended chosen unit weights

Mix type	Unit weight	CUW %
Fine-graded	CA LUW[1]	60 - 80
Coarse-graded	CA LUW	95 to 105
SMA	CA RUW	110 to 125

[1] CA = Coarse aggregate.

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

A.4 Loose and rodded unit weight voids

The loose unit weight voids are derived from the loose unit weight, and the bulk relative density of the coarse aggregate as presented in Eq. A.2. Similarly, the rodded unit weight voids are derived from the rodded unit weight, and the bulk relative density of the coarse aggregate as presented in Eq. A.3. Typical ranges of voids are presented in Table 31.

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

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

where:

V_{LUW} = Loose unit weight voids
V_{RUW} = Rodded unit weight voids
LUW = Loose unit weight
RUW = Rodded unit weight
RDA = Bulk relative density of aggregate

Table 31: 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 32: Characteristics of 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 ≫ RUW • Coarse fractions carry load • Remaining voids filled with mastic

A.5 Aggregate packing analysis

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

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

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

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

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

Table 33, Table 34 and Table 35 show the control sieves and recommended aggregate ratios for fine-graded, coarse graded and SMA mixes, respectively.

Table 33: Control sieves for fine-graded mixes

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

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

Table 34: Control sieves for coarse-graded mixes

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

Table 35: Control sieves for SMA mixes

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

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

A.6 Effects of aggregate ratios on VMA

Table 36 and Table 37 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 36: Recommended ranges for aggregate ratios in fine and coarse mixes[1]

NMPS (mm)	CA (coarse-graded)	New CA ratio	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
28	0.70-0.85			
20	0.60-0.75			
14	0.50-0.65			
10	0.40-0.55			
7,1	0.35-0.50			
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 37: Recommended ranges for aggregate ratios in SMA mixes

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

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

Table 38 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 39.

Table 38: Effect of VMA - changes in aggregate ratios

	Fine-graded	Coarse-graded	SMA
CA	increase	increase	increase
FA_c	decrease	decrease	decrease
FA_f	decrease	decrease	decrease

Table 39: Change in value of Bailey parameters to produce 1% change in VMA

	Fine-graded	Coarse-graded
CA	0.35	0.20
FA_c	0.05	0.05
FA_f	0.05	0.05

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

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

A.7 Procedure to blend aggregates

The designer needs the following information:

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

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

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

Steps for blending aggregates using the Bailey method:

1. Conduct three laboratory tests on all aggregate fractions; (a) grading (b) BRD of aggregates, and (c) Unit weights - LUW, RUW.
2. For aggregates designed to obtain fine-graded mixes, select CUW (%) based on coarse aggregate LUW (Table A - 2)). On the other hand for aggregates designed to obtain SMA mixes the CUW is based on coarse aggregate RUW.
3. Determine the unit weight (LUW or RUW) contributed by each coarse aggregate according to the desired proportions (by volume) of coarse aggregate (*contribution = percent coarse aggregate* x *chosen unit weight*).
4. Determine the voids in each coarse aggregate according to its corresponding CUW and contribution by volume. Then sum the voids contributed by each coarse aggregate.
5. Determine the unit weight (LUW or RUW) contributed by each fine aggregate according to the desired proportions (by volume) of fine aggregate.
6. Determine the initial blend percentage by mass of each aggregate. Divide the mass of each aggregate fraction by the mass of the total aggregate blend.
7. Determine the amount of material passing 0,075 mm sieve contributed by each aggregate fraction.
8. Determine the amount of filler required, if any, to bring the percent passing the 0,075 mm sieve to the desired level.
9. Once the desired amount of material passing 0,075 mm sieve is achieved, adjust the final blend percentages (by mass) of fine aggregate fractions. In this step the blend percentage of coarse aggregate is not changed.
10. The final blending percentages (by mass) and aggregate ratios are determined and checked against Bailey requirements.

Appendix B

Principles of aggregate proportioning 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 – 4 %.

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, the design procedure developed in the USA, initially by NCAT with NAPA support and ultimately published by AASHTO[5] is recommended, as it incorporates the fundamental principles of a binary system (aggregate and mastic) developed by Zichner and offers a sound procedure for attaining the requisite aggregate configuration, with contiguous stone on stone contact.

An alternative approach is offered by the Bailey method (see Appendix A whereby a Chosen Unit Weight (CUW) of 110 – 125 % of Rodded Unit Weight (RUW) is selected. While this will in all probability assure contiguous contact in the coarse aggregate, the approach is empirical and does not address fundamental principles of composition of SMA.

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.

[5] See section B.3 Design method

B.3 Design method

It is not the intention here to present a comprehensive design method. Rather the reader is referred to the following documents which present a comprehensive design procedure and set of standards, based on this binary method of design:

- AASHTO R46: *Standard Practice for Designing Stone Matrix Asphalt (SMA) using the Superpave Gyratory Compactor™ (SGC)* and
- AASHTO M 325-08 (2021): *Standard Specification for Stone Matrix Asphalt (SMA)*.