



## User Guide for the Design of Asphalt Mixes

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(A practical guide for the application of the general procedure for designing asphalt mixes as set out in Sabita Manual 35)



excellence in bituminous products

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# Sabita manuals and DVDs

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

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## SCOPE OF THE DOCUMENT

The purpose of this document is to serve as a practical guide for the application of the general procedure for designing asphalt mixes as set out in Sabita Manual 35: *Design and Use of Asphalt in Road Pavements*, published in June 2016 and revised in February 2020. The document strives to explain the design principles described in Sabita Manual 35 to ensure that designs prepared are optimal for their application in terms of resources of constituent materials, traffic and climate.

This guide emphasises key content of the manual, cautions on limitations of some procedures and compliance requirements and gives guidance on supplementary methods or procedures to enhance the design procedure. Also, where appropriate, some background is given on the principles adopted in the method for guidance, especially to practitioners with limited experience in the design of asphalt mixes.

While the focus of this document will be on the general method of design of asphalt, it will also draw the user's attention to special considerations that should be taken into account when designing warm mix asphalt, recycled asphalt, thin asphalt wearing courses, open graded asphalt, EME (i.e. high modulus asphalt) and stone-mastic-asphalt (SMA).

It should be emphasised that this User Guide serves as a **companion document to Sabita Manual 35**, which is available from Sabita (+27 21 531 2718 or [www.sabita.co.za](http://www.sabita.co.za)).



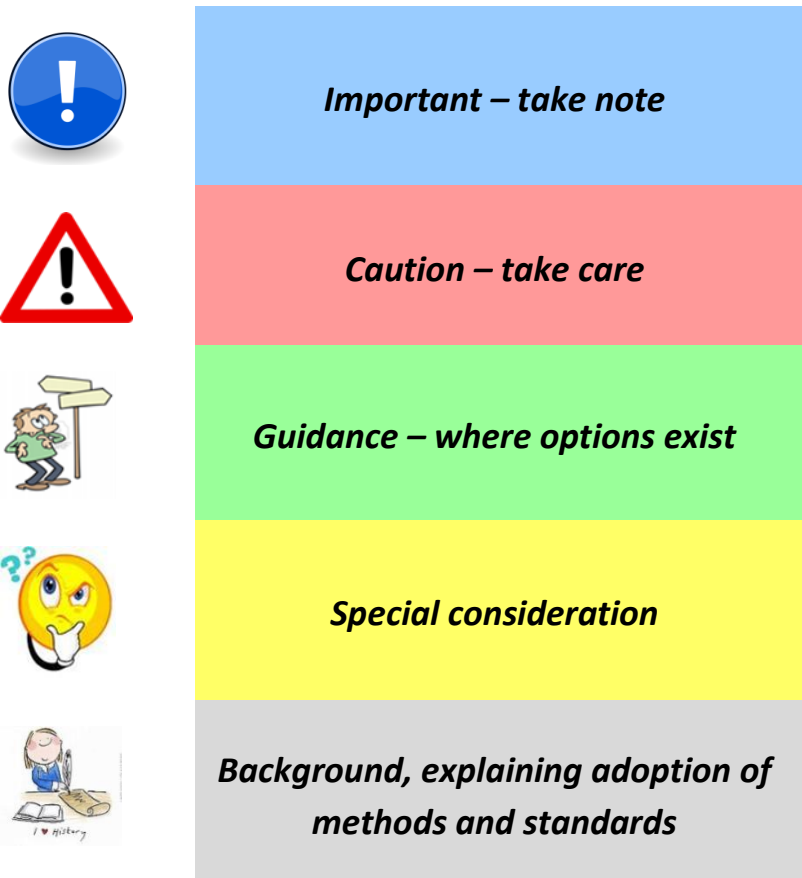
# 1. INTRODUCTION

The purpose of Sabita Manual 35 was to establish a standard, common base for the design of asphalt mixes in South Africa. This manual reflects a move towards performance-related design procedures in line with international best practice.

A key feature of Manual 35 is the adoption of 4 levels of sophistication in the mix design procedures, related to the status of the road under consideration in terms of risk aversion. At the lowest level (IA) a largely volumetric approach is presented, whereas at the highest level (III) advanced performance tests are carried out in order to ensure that the mix will perform adequately in terms of its response to loading and resistance to damage.

Also in this method the user is encouraged to come to grips with the determination of optimal aggregate structure for a particular application, rather than adherence to stereotype gradings. Gradings are deemed to be more appropriate to quality assurance measures, once the aggregate/binder system has been optimised.

To assist the reader in using this guide, certain sections are highlighted in a particular manner to draw attention to important content and to give additional guidance:



Useful references are indicated in the right hand side margin to afford the reader ready access to more detailed information on a particular topic (as illustrated here).

Sabita Manual 35



## 2. MIX DESIGN PROCESS

The main aim of asphalt mix design is to find a cost-effective combination of binder and mineral aggregate, that:

- is workable in the field;
- contains sufficient bituminous binder for durability and resistance to fatigue; and
- has a suitable aggregate arrangement providing:
  - structure to the mix; and
  - space between particles to accommodate sufficient bituminous binder without flushing and / or bleeding.

In the case of a **wearing course**, the objective is to provide a layer that is waterproof (with the exception of porous asphalt) and meets functional requirements such as friction, noise attenuation and comfort.

The intention of this user guide is to assist mix designers in achieving these aims through providing a road map to a systematic approach in which each step or phase is progressively addressed to reach the design goal.

In Manual 35 the concept adopted is that mix properties should be defined and evaluated in terms of the loading and environmental conditions that the asphalt material will be subjected to in service. Consequently, test parameters determined during the mix design phase should have a direct relation to the performance of the material in the pavement structure.

The new approach adopted in Manual 35 is to do away with over-specification and to have **a single, validated criterion for each performance requirement**. In terms of this approach a single test is described for each performance indicator.

Not only does this simplify the design process, it allows direct comparison of the performance characteristics of different mix designs.

As shown in Figure 1 the mix design process follows a logical sequence, starting with the **evaluation of the design situation** i.e. the external factors that will influence decisions to meet specific **design objectives**.

Once these “input factors” have been identified the actual selection of **component materials and their proportioning** commences. A first decision in this respect is the selection of the mix type, i.e. a stone or sand skeleton aggregate configuration. Following this step, component materials are selected, given considerations of availability, cost and performance requirements. The final proportioning of these component materials is determined through a formal design process, the complexity of which is determined by the application. This process ranges from fairly straightforward volumetric design processes (Level IA) to advanced performance testing to determine response and damage characteristics (Level III). The final “output” is a project specification which meets the various requirements optimally.

Each of the steps in the design sequence is dealt with in some detail below.

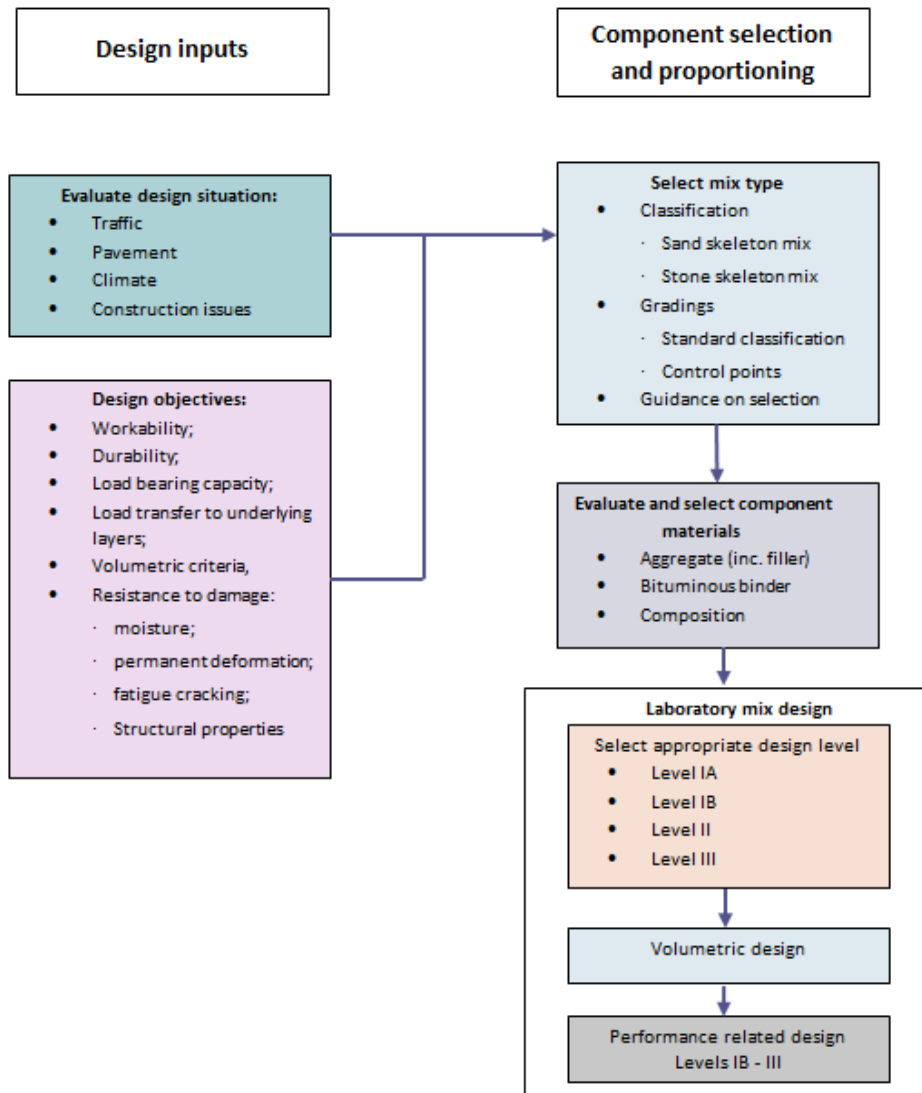


Figure 1: Mix design process

## 2.1 EVALUATE DESIGN SITUATION

This phase of the design process is represented in Figure 2.

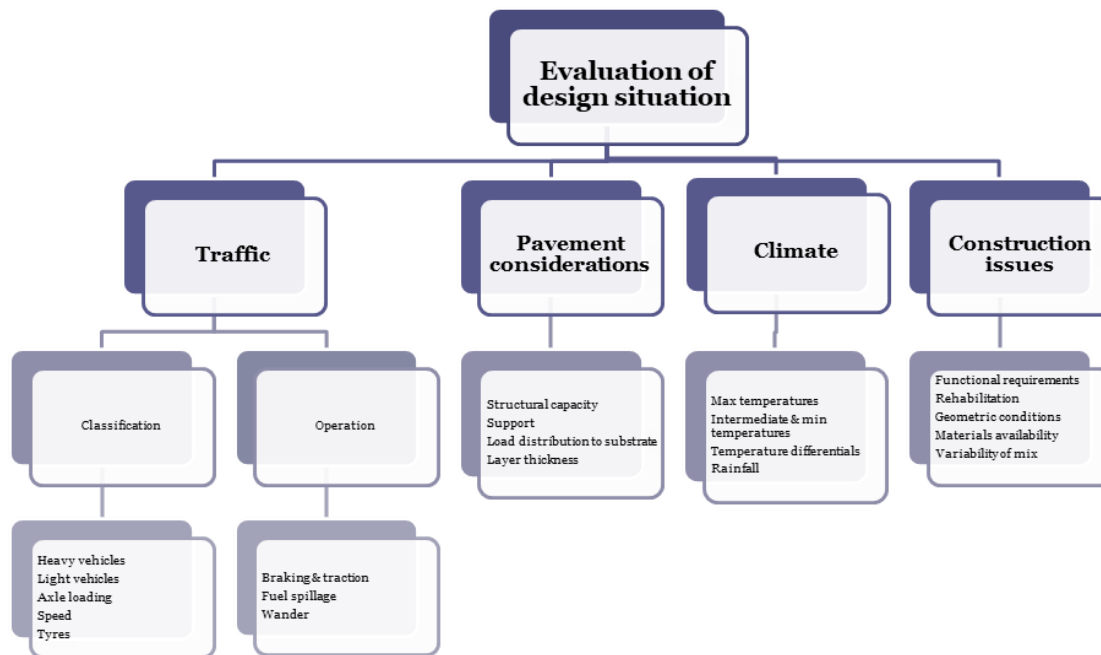


Figure 2: Evaluation of design situation

Manual 35 deals with each aspect in detail; however, the user would do well to pay particular attention to the following aspects.

### 2.1.1 TRAFFIC

#### Classification

#### Heavy vehicles

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

Since an asphalt mix gradually stiffens over time due to binder hardening, it is especially the intensity of axle loads applied to the newly constructed pavement that will test the resistance of a layer to permanent deformation. Early intense loading, such as that which occurs on a rehabilitated pavement, is more severe than the less intense loading, which normally occurs on a newly constructed road.



Designers should also be aware that early trafficking on newly laid asphalt might also cause premature distress and time for cooling of the layer should be a major consideration for determining when the layer would be opened to traffic.

For the purposes of mix design, heavy traffic intensity is evaluated using traffic classes, as discussed below.

## Light vehicles

The loads imposed by light vehicles (e.g. small passenger vehicles) are too gentle to induce damaging stresses and strains in well-designed pavement layers which cater adequately for heavy vehicles.

However, the volume and speed of light traffic need to be taken into account when considering functional properties such as:

- friction;
- noise reduction; and
- riding quality

Manual 35 gives guidance on mean profile depth – MPD requirements for mixes placed on roads where the speed of light traffic exceeds 60 km/h. It also mentions the need for mixes with improved noise reduction properties in urban areas where the volumes of light traffic are high.



Also, in cases such as residential areas where the traffic is predominantly light, densification of the layer under the action of light traffic is unlikely to be significant. Consequently initial 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.

In this respect the reader would be well advised to consult Sabita **Manual 27: Guideline for thin layer hot mix asphalt wearing courses of residential streets.**

Sabita Manual 27



## Heavy vehicle axle loading

Whereas in the past, traffic demands of > 10 million ESALs were considered very high, these volumes can readily be exceeded at present, particularly on bus rapid transit routes where, due to the frequency of vehicles, high axle loadings and narrow tracking, volumes may readily exceed 30 million ESALs and even reach 100 million ESALs. In such cases, particularly for asphalt bases, special consideration should be given to mix type, in terms of aggregate packing to carry and distribute loads and bituminous binder type and concentration in terms of resistance to viscous flow and fatigue as well as the ability to spread or distribute the loading to underlying layers.

Axle loads are limited to maximum values by law and 80 kN is currently used as a standard in pavement design calculations.



The designer should note, though, that owing to lack of comprehensive law enforcement a fairly large proportion of axle loads operating on SA roads exceed this value of 80 kN.

## Vehicle speed

It should also be noted that *both the volumes and speed* of, particularly, heavy vehicles will have an influence on the tendency of an asphalt layer to deform (i.e. shove or rut). This aspect is accounted for in the classification of binder type in terms of its performance grade. (See 0Table 3)

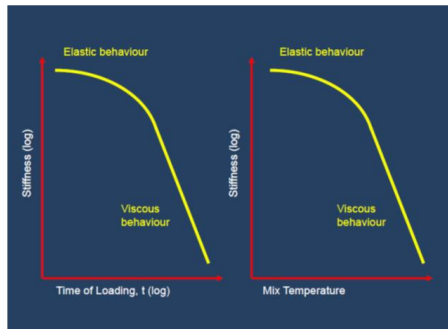
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The user should take note that it is advisable that traffic loading intensity be considered in conjunction with operating speed, since asphalt displays *visco-elastic behaviour*.



At high vehicle speeds (i.e. short loading times) asphalt displays higher stiffness and behaves elastically. Consequently it will recover (rebound) with little permanent deformation once the load is removed. As speeds reduce (i.e. longer loading times) viscous behaviour sets in, with a reduction of stiffness and a greater extent of permanent deformation. The effect of temperature is similar; at low temperatures, the asphalt stiffness is higher and the material tends to behave elastically. As the temperature of the layer increases, stiffness is reduced and more viscous behaviour sets in leading to higher permanent deformation. This concept is illustrated in Figure 3.



**Figure 3 Visco-elastic behaviour of asphalt**

Dynamic pavement models as well as strain measurements taken at various vehicle speeds have shown that tensile strains at the bottom of the asphalt layer may decrease by as much as 50 % as vehicle speeds increase from creep speed to about 80 km/h, or vice versa.

Mixes designed for low speed situations, e.g. climbing lanes, intersections or any other condition where heavy vehicle speeds are predominantly less than approximately 30 km per hour, require special consideration.

The designer should note that the Design Levels (IA – III) listed in the last column of Table 3 relates to the traffic loads and volumes (expressed as E80s) **over the service life of the asphalt pavement**. The intention is to adopt an appropriate level of sophistication in design procedures to mitigate exposure to the risks associated with structural damage.

## Tyres

Tyre construction, inflation pressures and tyre loading all play a significant role in rutting and fatigue cracking in asphalt. It should be noted that, ignoring tyre wall stresses, contact stresses at the tyre pavement interface is of the same order of magnitude as the inflation pressure.

By using fewer tyres modern trucks are exerting much higher contact stresses. The high pressures associated with the increased use of super-single tyres clearly place greater stress on the asphalt layers located in the upper zone of the pavement, which is more exposed to the direct influence of the wheel loads.

Figure 4 below illustrates that, for a specific tyre load (18 kN), the vertical contact stress increases significantly as the inflation pressure is increased. Conversely, Figure 5 illustrates that for a given inflation pressure, the wall exerts a significant pressure on the surface of the pavement as the wheel load increases.

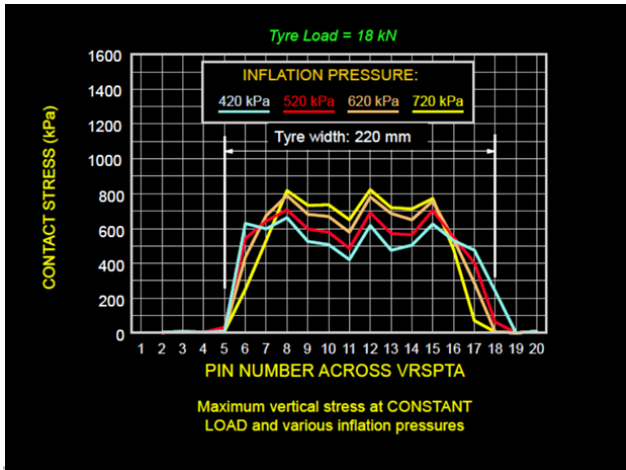


Figure 4: Tyre contact stress at constant load

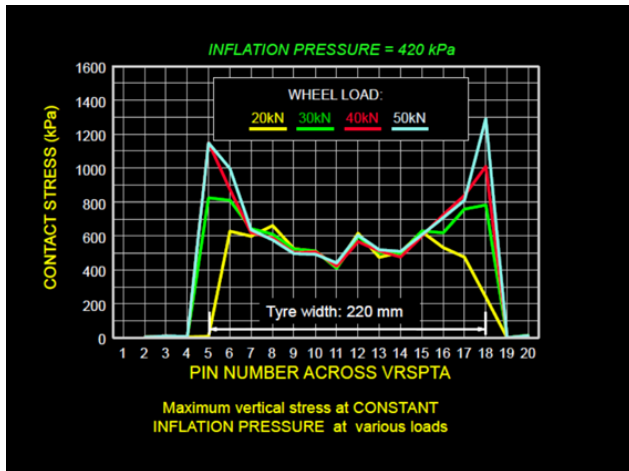


Figure 5: Tyre contact stress at constant inflation pressure



The designer should take note that, whereas nominal values of traffic volumes are adopted in the design of asphalt wearing course and base layers, tyre type and pressures give rise to a large range of actual imposed stresses on the upper pavement layers, which will adversely affect the performance of the asphalt layers.

### Operation

#### 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 therefore not be appropriate for intersections.

#### Fuel spillage

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

## Wander

As traffic wheel paths tend to be distributed laterally within the lane width, the extent of spread or wander will have an effect of the nominal intensity of loading. Areas of narrow wander i.e. more concentrated loading, occur typically on bus rapid transit routes and at work zones with lane closures.

### 2.1.2 PAVEMENT CONSIDERATIONS



It should be noted as a fundamental point that the design of asphalt mixes cannot proceed in isolation. By necessity, integration of the mix design process and structural design of the pavement is a basic principle of sound pavement engineering.

Consequently, the design procedure in Manual 35 strives to establish a relationship between laboratory tests and the performance characteristics of the mix.

While this aspect is fairly comprehensively covered in Manual 35, a number of selected matters are highlighted in this guide to encourage the designer to be mindful of the fact that designing an asphalt mix comprises more than developing a composition of components that merely comply with volumetric requirements; the goal is to devise a mixture composition that will respond adequately to the dynamic loads and environmental stresses imposed on it.

The process of transition to a performance related specification for bituminous binders reached a benchmark with the publication by SABS of SATC 3208:2019, a technical specification for Performance Grade (PG) specifications for bitumen in South Africa. In this specification a number of rheological parameters are defined to facilitate the optimal selection of a binder for a particular application of an asphalt mix. It is foreseen that shortly binders will be selected according to this PG framework to ensure adequate performance of asphalt pavement layers in terms of imposed loading and the prevailing environment.

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Sabita Technical  
Guideline on PG



### *Structural capacity*

As indicated above the design method provides for standard routines to design asphalt to cater for design traffic in excess of 100 million E80's. To this end the design routines have been divided into four procedures with increasing complexity as the traffic category increases. The reason for this approach is to ensure that risks associated with inadequate performance are reduced as traffic increases, in order to mitigate the severely adverse effects of premature distress of the pavement layers.

### *Support*

Under most conditions the behaviour of the asphalt layer is predominantly influenced by the properties of the layer immediately below it. In the case of asphalt wearing courses, the quality of the base course would have a marked influence on its composition and, ultimately, its performance; in the case of an asphalt base, the subbase / substrate system would be the influencing factor.



For an asphalt layer with a thickness of less than about 50 mm, flexing under loading is influenced almost completely by the stiffness of the support layer(s) and, to a lesser extent, by the properties of the asphalt layer itself. When the asphalt layer is thicker than about 80



mm, the influence of the support layer decreases somewhat and the amount of flexing taking place in the asphalt layer is moderately influenced by the properties of the asphalt layer itself.

The amount of flexing gives rise to tensile strains which may induce fatigue fracture

In the case of rutting, it is reasonable to consider that this distress type is primarily a mix design issue to counter excessive shear deformation, whereas fatigue is influenced by both the pavement structure and a mix components and proportions.

Apart from these performance aspects, support conditions will also influence the ease of construction of the asphalt. Although compactability of a mix is primarily affected by its workability at paving temperatures, it is also significantly influenced by the stiffness of the substrate.

In the case of thin (< 30 mm) asphalt layers, the final riding quality may also be determined, to some extent, by the properties and evenness of the layer on which the asphalt is being laid.



Laying of asphalt on very weak and / or variable support conditions would generally NOT reflect sound practice.

### *Layer thickness*

While layer thickness is primarily a structural design element, Manual 35 gives important guidance of minimum layer thickness in relation to aggregate size or, conversely, maximum aggregate size in relation to predetermined layer thicknesses.



This aspect is often overlooked and is a very important consideration to assure the uniformity and integrity of the completed layer.

### **2.1.3 CLIMATE**



Climatic conditions have a marked influence on the behaviour and performance of asphalt layers, particularly as a result of the rheological behaviour of bituminous binders.

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Factors related to climate to consider are:

- Temperature
- Rainfall
- Moisture sensitivity of substrate

### *Temperature*

#### **Maximum temperature**

Elevated temperature is a key factor influencing plastic deformation (rutting and shoving) of asphalt layers. Climatic conditions, in which high asphalt temperatures are likely to be experienced for large percentage of time, require special attention to counter permanent deformation. Consideration of the maximum temperature may influence the selection of:

- aggregate configuration (skeletal structure and maximum aggregate size);
- aggregate type; and
- binder type.

As regards the latter, the new PG system for bituminous binders will facilitate the selection of the optimal binder for a specific location.

### Intermediate and minimum temperatures

Low temperatures associated with a particular zone have a significant influence on both asphalt fatigue and low temperature fracture. In this case the choice of binder again plays a significant role; hence selection of the optimum binder grade is of crucial importance. The reason for this is that fatigue and low temperature fracture occur as a result of loss of cohesion within the binder, whereas the influence of aggregate packing is somewhat less or secondary.

A binder property required for adequate resistance to fatigue distress is mainly toughness, whereas for resistance to low temperature fracture requires adequate stress relaxation properties. Both these parameters will be addressed in the new PG binder specification.

### Extreme temperature differentials

High annual differentials in operating temperatures requires a fine balance in the design to achieve both good rutting resistance during prolonged periods of elevated temperatures while also protecting the mix against fatigue and low temperature fracture during extreme low temperatures during, say, winter months.



In such cases extreme care should be taken in the selection and composition of constituent mineral aggregate and binder grade. The designer would be well advised to study recorded sound practice in such regions as well as to consider consulting an experienced professional in such matters.

### Rainfall

Manual 35 does indicate that designing mixes for projects located in areas with high and/or prolonged rainfall requires special attention to limit the permeability of the layer and to guard against the increased potential for stripping and to achieve durability.

Rainfall considerations may thus influence the choice of aggregate type, filler type, binder type and selection of aggregate packing.



Asphalt wearing courses located in areas of high (and prolonged) rainfall would also require special attention to the skid resistance characteristics of the layer.

### Moisture sensitivity of substrates

In many instances, relatively thin asphalt surfacings not only distributes the traffic loading (albeit to a limited extent), it provides a moisture barrier for underlying granular materials which are often moisture-sensitive. Therefore, in such cases, wearing course layers should be designed to be impermeable to water in the liquid phase.

#### 2.1.4 CONSTRUCTION ISSUES



In this section a number of matters associated with the finished product are raised for consideration by the designer.

## Functional requirements

In addition to satisfactory structural performance of asphalt layers, wearing courses, which provide the interface of wheels and the pavement, should yield acceptable functional performance. The functional performances indicators include:

- Surface texture for:
  - adequate skid resistance
  - surface water draining during rain: and
  - limited noise generation potential (especially in urban areas);
- Riding quality;
- Noise generation; and
- Visibility during rain

### Surface texture

The performance rating of various mix types in terms of skeletal structure (i.e. sand or stone skeleton) is given in Table 7 of the Manual 35, from which it is clear that stone skeleton mixes are, in general, more suitable to provide skid resistance and / or abate road noise. While, as a rule, this may be valid, it should be borne in mind that sand skeleton mixes, if carefully designed can provide adequate skid resistance, particularly where operating speed is low.

Skid resistance is primarily influenced by micro-texture and macro-texture of the aggregates in the road surface. The texture of the road surface influences friction developed between the tyre and asphalt surface to prevent skidding. Table 4 and Figure 1 of Manual 35 define classes of texture and their functional characteristics, respectively.

Sabita Manual 35 recommends the achievement of high macro texture (or high mean profile depth - MPD) for mixes placed on roads where the speed of light traffic exceeds 60 km/h.



Manual 35 is somewhat critical regarding the practice of rolled-in-chips on continuously graded asphalt to improve surface texture. It states clearly that this practice is not recommended in view of the possible adverse effect on mix performance in terms of durability and permeability. It is suggested that this practice should only be adopted where alternative designs and measures are not feasible and where the asphalt manufacturer undertakes to design the mix with due consideration of these effects

### Riding quality

A lack of evenness obtained after paving may accelerate the rate of deterioration of riding quality for a number of reasons, resulting ultimately in diminished pavement serviceability and an increased vehicle operating costs. The designer should ensure that the designed mix is not susceptible to shoving during paving and, especially, compaction by rollers, which will lead to an uneven finish and possibly built-in cracks due to excessive horizontal movement of the mat during compaction.

A number of precautions to be taken during the paving operation to ensure good riding quality of the layer are given in Sabita Manual 5: *Guidelines for the manufacture and construction of hot mix asphalt.*

Sabita Manual 5



The designer should note that riding quality is often affected by lift/layer thickness. Increased layer thickness tends to have a negative effect on riding quality due to increased

movement under the rollers during compaction. This effect may be aggravated by irregular or haphazard rolling patterns and high paver-drag.

For optimum riding quality, thick asphalt base layers may be split into two applications. In general, layer thicknesses in excess of 70 mm can adversely affect riding quality.

### Noise generation



The most effective means of reducing road noise is through the use of porous asphalt wearing course. Porous asphalt has a void content in excess of 20%, with the voids interconnected. The high void content of porous asphalt, combined with layer thicknesses of at least 40 mm, serves to attenuate road noise generated by vehicle tyres - and to a certain extent by vehicle engines - by absorption. The noise reduction relative to dense bituminous mixes equates to 50%.

Porous asphalt contains a high proportion of coarse aggregate and limited amounts of fines and filler. It is manufactured in a conventional asphalt plant, transported to site, laid and compacted on a sound and impermeable underlying layer.



It should be noted that legislation requires that predefined noise level criteria may have to be met, depending on the location of the road relative to the nature of the adjacent environment e.g. rural residential, urban residential, industrial. This may result in limitations on the type of surfacing to be used to comply with the noise limits set for a particular district. Porous asphalt, being a cost-effective alternative to noise barriers, offers both road users and urban residents a highly competitive means of addressing the growing concern for the environment in densely populated areas.

The design of porous asphalt mixes is dealt with in Sabita Manual 17: *Porous asphalt mixes – design and use*.

### Visibility during rain



To improve visibility during heavy rains by reducing backsplash from especially trucks, porous asphalt should be considered on important routes. The high percentage of interconnected voids provides drainage paths for water to the outer boundaries of the surfacing. The design is covered in the above mentioned Sabita Manual 17.

Sabita Manual 17



### Rehabilitation

Designers should note that partial or dispersed rehabilitation measures result in the construction of numerous joints both at the surface and in depth which would render the project more susceptible to water ingress. Consideration should therefore be given to the rehabilitation of larger areas resulting in fewer joints, and appropriate method statements should be specified.

### 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. To counter distress in such areas would require careful consideration of aggregate packing, binder type and mix proportions.

The designer should take into account that some difficulty may be expected in achieving specified finish tolerances and compaction at intersections, steep grades, and on highly flexible substrates; hence maintaining a minimum layer thickness would require special attention.

## Wander

The degree of wander or lateral spread of actual traffic paths in traffic lanes can have a significant effect on rutting and fatigue distress. On high speed roads, with wider lane widths, the degree of wander is normally greater 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. (See Figure 6)

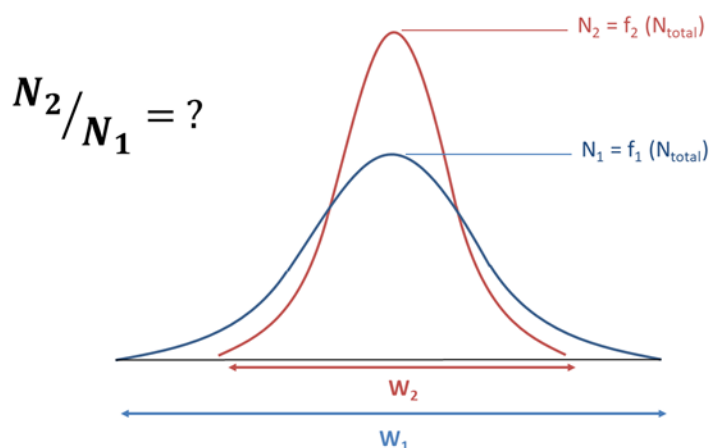


Figure 6: Effect of wander on peak repetitions

As an example, Figure 7 illustrates the severity of tensile strains associated with the extent of wander (in terms of a standard deviation). From this figure it can be deduced that a reduction in wander could lead to a proportional increase in tensile strains in the asphalt with its associated effects on layer distress.

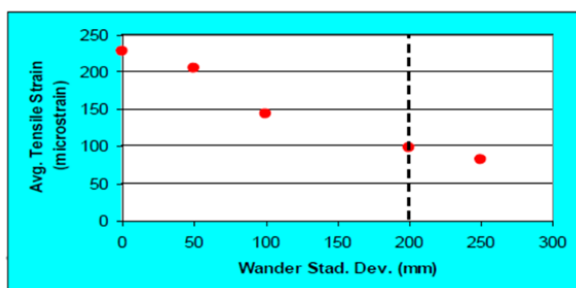


Figure 7: Effect of traffic wander on layer strain

## Materials availability



Certain mixes are more forgiving than others in terms of the effect of variations in aggregate properties and grading on the mix performance. While these characteristics are dealt with more fully in the design procedures, at this stage it should be borne in mind that marginal or variable aggregates should not be used in mixes that are highly dependent on aggregate interlock, such as Stone Mastic Asphalt (SMA).

Furthermore, if aggregates are unlikely to provide sufficient deformation resistance owing to their quality and variability, a binder of higher viscosity or a modified binder should be selected to reduce the potential for segregation and to increase the stability of the mix.

The availability of aggregates, filler and bitumen of the required quality should be evaluated before project specifications are finalised. Such evaluation at an early stage may lead to innovative practice in the interest of cost-effectiveness or may alert the client and tenderer to additional costs that may be incurred through transport or special manufacturing processes needed to produce the desired quality of materials in the mix;

The aggregate types available from commercial sources and bitumen materials commonly used for asphalt production in South Africa are given in Table 5 and Table 6 of Sabita Manual 35, respectively.

### Mix variability

Variability of the quality of the paved layer can derive from two main factors:

1. variability of the component materials – especially aggregate and natural filler
2. processes associated with the construction of the layer.

One such case is segregation of aggregates during construction which results in variability of the binder content and aggregate particle size distribution and, hence permeability and denseness of the layer.



The finer fraction of the asphalt mix will yield binder contents higher than the mean content, while a coarser portion results in lower binder content. Segregation may also result in variation of density and voids, as well as the overall performance of the mix.

Segregation may be worsened especially during loading and paving of large aggregate mixes (See SABITA Manual 5: “*Guidelines for the manufacture and construction of hot mix asphalt*”).

Sabita Manual 5



### Component materials



A key feature of a successful design process is assurance that the design is based on representative samples of component materials available during the construction of a project. This success depends on two factors:

1. The samples used for carrying out the laboratory design are representative of the component material stock;
2. Materials delivered into stockpiles at the manufacturing facility (plant) are consistently essentially the same as those on which the design is based.

An essential component of such a quality management process is the construction and use of aggregate stockpiles aimed at assuring consistency and conformity of the material.

While it is not the intention of this document to deal with this issue, the designer would do well to consult Sabita Manual 5: *Guidelines for the manufacture and construction of hot mix asphalt*”.

Sabita Manual 5



Since reclaimed asphalt is now increasingly being used as a significant component of asphalt mixes, consultation of Sabita Manual 36: *Use of reclaimed asphalt in the production of asphalt* is recommended.

Sabita Manual 36 /  
TRH21



## Construction processes

Designers and those responsible for construction should also be aware of the variation that occurs in the mix properties across the width and length of a paver-mat during construction. This variation often occurs due to:

- the mix segregating within the paver box
- differences in temperature of truck loads delivered to the paver
- drainage of the binder in the truck body during haulage
- differential cooling of the paved mat

As a consequence, areas that appear visually coarse, fine or rich or lean in relation to uniform sections, should be regarded as distinct and should be designated and tested separately as discrete uniform sections.

### 2.2 ESTABLISH DESIGN OBJECTIVES

Apart from establishing the performance characteristics of an asphalt layer in a specific pavement configuration and location, certain properties of the mix needs to be considered to ensure that it is inherently sound and that its component materials are of adequate quality and optimally proportioned to facilitate construction and meet the performance requirements for an extended period. These objectives are listed in Figure 8.

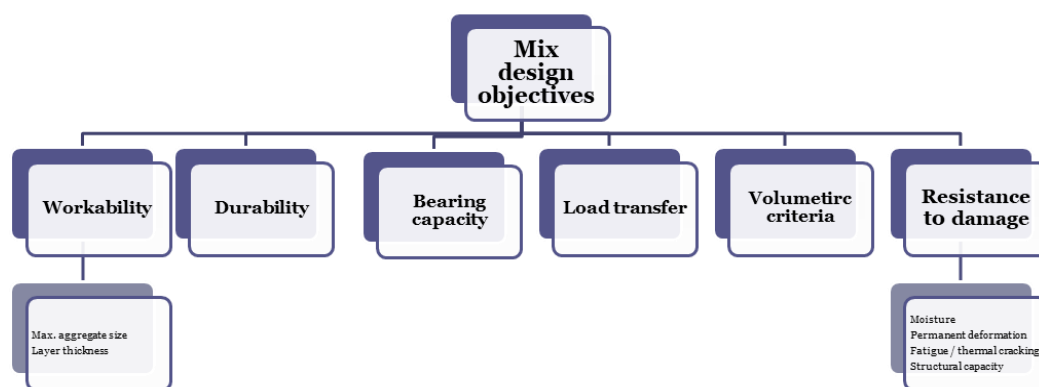


Figure 8: Mix design objectives

#### 2.2.1 WORKABILITY



Workability of a mix is an important performance characteristic and refers to the ease of handling, placing and compaction of a mix. The designer should take note that the achievement of whatever performance criteria set for a particular asphalt layer may be severely compromised by poor workability, i.e. constructing a layer with defects in which the target performance criteria are not consistently met.

A number of factors related to the component materials and mix temperature that will influence workability are listed in section 5.2.1 of Manual 35 and the designer should give these factors serious consideration in selecting his component materials while ensuring that ideal temperatures during manufacture and, especially, construction are maintained.

Sabita Manual 35



However, a factor that is not always given due consideration, is the relationship between maximum aggregate size permitted in relation to the specified layer thickness, as discussed below.



Firstly this aspect should NOT primarily be approached as a restriction of **minimum** layer thickness for a selected maximum aggregate size; rather it should be approached as a **maximum aggregate size** permitted for a predetermined layer thickness, which is based on pavement design considerations.

It should be borne in mind that a specified layer thickness is not necessarily consistently achieved, due to e.g. substrate unevenness or paving quality. Consequently there may be a significant area of the layer over which there is a deficiency in layer thickness and where the relationship of maximum aggregate size and layer thickness is compromised. Hence target ratios should account for this.

### *Maximum aggregate size*



In this method maximum aggregate size is (arbitrarily) defined as the Nominal Maximum Particle Size (NMPS) which is designated as:

***One sieve size larger than the largest sieve to retain a minimum of 15 percent of the aggregate particles***

Typically the selected NMPS for the asphalt mix is determined by:

- Location of the asphalt course in pavement;
- Proposed compacted thickness of a layer being paved<sup>1</sup>, and
- Functional requirements of the asphalt layer.

Except for UTFc's and porous asphalt, it is generally accepted that the NMPS should not exceed one third of the layer thickness to ensure compactability and to limit segregation during paving. As an example, for a 50 mm asphalt layer, the NMPS should not exceed 14 mm or for a 30 mm layer the NMPS should not exceed 10 mm.

### *Layer thickness*



Recommended minimum layer thicknesses for NMPS are listed in Manual 35 (Table 2). As this may be misleading, suggesting that minimum layer thicknesses should be met, the values are transcribed in Table 1 below as guidance and to accentuate the fact that **layer thickness in general is determined by pavement design considerations such as cover and stress distribution and that aggregates should be selected to tie in with these aims.**

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<sup>1</sup> The term layer here refers to the lift thickness being paved and compacted. The total layer thickness may consist of more than one lift so constructed.



**Table 1: Maximum NMPS**

Layer thickness range (mm)	Nominal Maximum Particle Size (mm)
20 - 25	7.1
30 - 35	10
40 - 50	14
60 - 80	20
90 - 120	28

### 2.2.2 DURABILITY

Manual 35 deals extensively with the issue of achieving a durable mix i.e. one that can resist (fatigue and low temperature) fracture due to e.g.:

- Hardening of the binder
- Disintegration of the aggregate;
- Stripping of the bituminous binder from the aggregate;
- Action of traffic and influence of temperature

A number of measures are suggested to ensure durability such as:

- An appropriate binder in adequately 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.
- Limiting the permeability of the mix



Manual 35 singles out **permeability as a key design factor** to ensure that the performance of asphalt is attained over a prolonged period, to ensure durability. It is suggested that special attention be paid to permeability of the mix during the design stage. Low permeability of a dense asphalt layer promotes long term durability and protects underlying layers from the ingress of water which may lead to premature distress due to increased flexibility of the substrate as a consequence of high moisture content.

Asphalt wearing course layers in South Africa are relatively thin (typically 40 mm); hence permeability is a critical factor particularly where 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.

Permeability of asphalt is a measure of the penetration of the mix by air, water and water vapour. 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 used to measure permeability in terms of experience gained.

The air permeability test adopted in Manual 35 is the standard described in Appendix C of TRH8 (CSRA 1987). (See Appendix B of this document). This method is currently being updated and will be published as a protocol ASP5 in Sabita Manual 39: *Laboratory Testing Protocols for Binders and Asphalt*

Factors that reduce permeability are:

- High binder contents with adequate film thickness;
- Dense aggregate packing;
- Dispersed rather than inter-connected air voids within the mix;
- Well compacted asphalt layers.



As regards voids dispersion, the designer is advised that gap- or semi-gap graded (sand skeleton) mixes are bound to have dispersed voids that do not readily offer flow paths for air or water.

### 2.2.3 BEARING CAPACITY

Since aggregates make up the bulk of the volume of asphalt mixes, the load bearing capacity of the asphalt layer is primarily provided by the aggregate. It stands to reason that the structural and functional performance of an asphalt mix in the pavement layer is greatly influenced by the physical properties and packing characteristics of the aggregate blend and, to some extent, the quality of the binder used.

### 2.2.4 LOAD TRANSFER

An important function of asphalt layers, both wearing courses and base layers is the capacity to transfer loads to underlying layers through spreading (or reducing) the intensity of the load on those layers.

The flexural stiffness of asphalt determines the extent to which traffic loading is spread to underlying layers. Relatively stiff asphalt is generally required for asphalt bases. However, where asphalt wearing courses are less well supported by bases in pavement structures with higher transient deflections, satisfactory performance may be better served by a lower stiffness, more pliant asphalt, to avoid traffic induced brittle cracking. The stiffness of asphalt is mostly influenced by:

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

### 2.2.5 VOLUMETRIC CRITERIA



The point of departure for mix design at any level (IA – III) is a volumetric analysis which allows for the (preliminary) assessment of some aspects of the mix properties and behaviour without necessarily conducting any additional mechanical tests on the asphalt materials.

In an asphalt paving mixture, asphalt and aggregate are blended together in precise proportions. The relative proportions of these materials determine the physical properties of the mix and, ultimately, how the mix will perform as a finished pavement.

In this section the generally used terminology to define critical volumetric properties and ratios will be presented. The process of optimising these various parameters will be covered in more detail in section 3.5.

## Terminology

### Air Voids

Air voids are small airspaces or pockets of air that occur between the coated aggregate particles in the final compacted mix. (See Figure 9) A certain percentage of air voids is necessary in all dense-graded mixes to allow for some additional pavement compaction under traffic and to provide spaces into which small amounts of bitumen can flow during this subsequent compaction. The allowable percentage of air voids (in laboratory compacted specimens) is between 2.0 percent and 4.0 percent for most dense mixes.

The durability of a dense asphalt layer can, in general, readily be linked to the air-void content. This is because the lower the air-voids, the less permeable the mixture becomes.



Too high a-void content provides passageways through the mix for the entrance of damaging air and water. Low air-void content, on the other hand, can lead to flushing; a condition in which excess bitumen is squeezed out of the mix to the surface.

### Percent Voids in the Mix (VIM)

The VIM is that part of the compacted mixture not occupied by aggregate or bitumen expressed as a percentage of the total volume. The VIM obtained in laboratory prepared design specimens thus gives an indication of the durability of the mix, its susceptibility to flushing as well as whether the mix can be compacted adequately in the field.

### Voids in the Mineral Aggregate (VMA)

VMA represents the air spaces that exist between the aggregate particles in a compacted paving mixture, including spaces filled with bitumen. (See Figure 9)

As such VMA represents the space that is available to accommodate the bitumen and the volume of air voids necessary in the mixture. The more VMA in the dry aggregate, the more space is available for the film of bitumen.

Factors that influence aggregate packing, and hence, the VMA of the aggregate blend are:

1. *Gradation* – particle size distribution is the **dominant aggregate packing factor**.
2. *Compaction method* – e.g. the compaction with a Marshall hammer orients the aggregate structure differently than with a gyratory compactor.
3. *Aggregate shape* – cubical particles tend to pack more densely than flat and elongated particles.
4. *Aggregate texture* – smooth particles pack more easily than those with rough surface texture (i.e. micro-texture).
5. *Strength* – aggregates of varying strengths pack differently due to degradation or break down depending on the compactive effort applied.



**Based on the fact that the thicker the bitumen film on the aggregate particles the more durable the mix, minimum requirements for VMA are specified which should be met to ensure that a durable bitumen film thickness can be achieved.**



**Increasing the denseness of the gradation of the aggregate to a point where minimum VMA values are not met, leads to thin films of binder and a lean, low durability mix. Therefore, economizing in binder content by lowering VMA is actually counter-productive and detrimental to layer quality.**

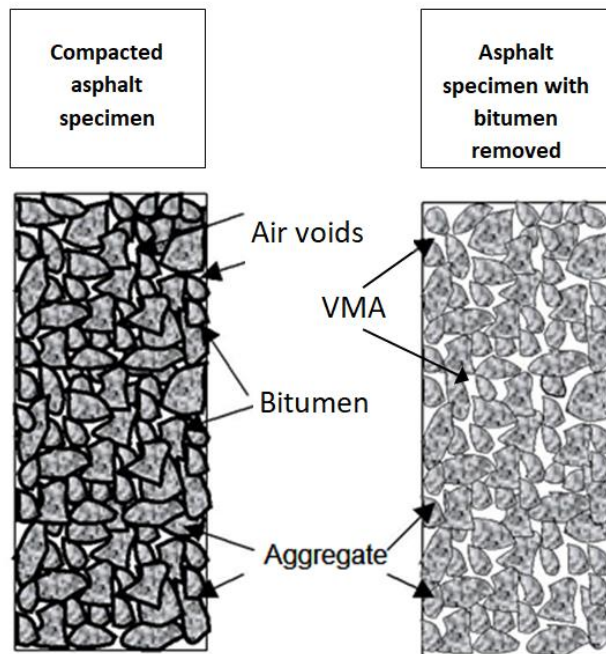


Figure 9: VMA in compacted specimen

### Percent Voids Filled with Bitumen (VFB)

The VFB is the percentage of voids in the compacted aggregate mass that are filled with bituminous binder.



The VFB property is important for two reasons. Firstly it is an indication of the cost effectiveness of the use of costly bituminous binder. For instance, having a high percentage of VFB may be indicative of high voids in the mineral aggregate, requiring a (unnecessarily) high volume of binder in order to reduce the VIM to an acceptable range and to ensure sufficient films of binder. Also, overfilling of voids in the coarse aggregate with fine aggregates or voids in the fine aggregate with mastic should be avoided as high VFB is known to have a strong relationship with poor rutting performance. Conversely, if the VFB is too low, there may not be sufficient bitumen to provide durability.

### Filler/Binder (F/B) Ratio

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



While the filler may serve the purposes mentioned above, the presence of too much or too “active” filler may cause the viscosity of the hot mastic during the mixing and compaction process to increase to such an extent that adequate compaction is not possible in the field. For these reasons limitations on the filler-binder ratio is used in the design process.

## Bitumen Content



The proportion of bitumen in the mixture is a key volumetric parameter and must be accurately determined in the laboratory and then precisely controlled on the job. The optimum bitumen content of a mix is highly dependent on aggregate characteristics such as gradation and absorptiveness.

Particle size is closely related to bitumen content that is optimal for a specific mix. The finer the mix gradation, the larger is the total surface area of the aggregate and the greater the amount of bitumen required to coat the particles adequately. Conversely, because coarser mixes have less total aggregate surface area, they demand less binder.

The relationship between aggregate surface area and optimum binder content is most pronounced where filler (i.e. material passing the 0.075 mm sieve) is involved. Small increases in the amount of filler in a mix can literally absorb much of the binder, resulting in a dry, unstable mix.

The ability of the aggregate to absorb bitumen is critical in determining optimum bitumen content. Ultimately, it is the **effective** binder content, i.e. **the total binder content less the quantity absorbed** into the aggregate that provides the coating of aggregate and their bonding.

## Binder film thickness



Binder film thickness represents a computed average thickness of the effective binder coating the aggregate particles in the mix. The common method used to compute the binder film thickness is to divide the volume of *effective binder* by the estimated surface area of aggregates. While the method is purely empirical, it is the best available at present.

The surface area of the blended aggregate is important for the determination of a minimum binder content in the asphalt mix to achieve a suitable binder film thickness and, hence, to achieve a durable mix.

The finer the mix grading, the larger the total surface area of the aggregate and the greater the amount of binder required to uniformly coat the aggregate particles. The calculation of the specific surface area (SA) of the aggregate particle is section 4.9 of Sabita Manual 35.

A schematic of the basic volumetric properties used in asphalt mix design is illustrated schematically in Figure 22,

Sabita Manual 35



The volumetric design procedure described in Manual 35 is based primarily on the determination of the following properties of compacted specimens:

- bulk density;
- voids in mix (VIM);
- voids in the mineral aggregate (VMA)<sup>2</sup>; and

<sup>2</sup> In the design of stone mastic asphalt (SMA) the determination of an entity “voids in the coarse aggregate” (VCA) is also required.

- voids filled with binder (VFB) at a pre-defined level of compaction.

To determine these volumetric parameters it is necessary to understand and derive the following properties related to the components and components of a compacted asphalt specimen:

- Bulk density of the individual and combined aggregate fractions
- Maximum voidless density of the compacted asphalt
- Bulk density of the compacted mix
- Effective density of the combined aggregate
- Relative density of the binder
- Absorption of binder by the aggregate
- Effective binder content

A full list of density and volume parameters used in volumetric analysis is given in Tables 16 and 17 of Manual 35, respectively.

Sabita Manual 35



As mentioned before, VMA is an important parameter as inadequate VMA could lead to potential durability problems by not allowing sufficient space for the effective bituminous binder and hence adequate binder coating of the aggregate, while attempting to meet minimum void requirements.

VFB can provide useful information on whether or not the VMA will be saturated with binder at, especially, refusal density.

The designer should take note of a number of issues related to determination of volumetrics:



- It is very important that volumetric properties should be checked for consistency and reasonableness when evaluating the various parameters.
- Small errors in the BD of the fines and the total BD of the mix, as well as absorption properties of the aggregates, can result in significant errors in the calculation of the voids in the mix and hence the design binder content.
- Accurate density measurements are critical to the confident calculation of volumetric parameters.

Designers should also be aware of porous aggregate that can continue to absorb quantities of binder – especially the maltene phase – long after construction, which could lead to apparent binder hardening. Ideally, an alternative, less absorbent aggregate source should be located. However, should an alternative source not be available, binder contents on the higher side should be selected, and the consequences of increasing the binder content critically evaluated.

The need for different void specifications for slow and fast lanes on heavily trafficked roads should not be underestimated, as the densification under traffic will differ.

### 2.2.6 RESISTANCE TO DAMAGE

Damage to asphalt can occur as a result of either degradation of the material as result of limited durability of the layer and its constituent materials or arising from the effects of traffic loading in terms of permanent deformation (rutting) or fatigue.

#### *Moisture damage*

As mentioned, long term durability is promoted by low permeability of asphalt i.e. offering resistance to penetration of the mix by air, water and water vapour.

The degree of susceptibility to moisture damage is assessed by means of the modified Lottman test (ASTM D4867M-09). The test entails preparing job mix specimens compacted to a void content corresponding to void levels expected in the field. The set is divided into two subsets of at least three specimens each and of approximately equal void content. One subset is maintained dry while the other is partially saturated with water and moisture conditioned. The indirect tensile strength of each subset is then determined. The potential for moisture damage is indicated by the tensile strength ratio (TSR) of the wet subset to that of the dry subset.

### Rutting

Rutting is a two-phase process comprising:

- Densification, accompanied by a decrease in volume
- Shear deformation at constant volume (See Figure 10).

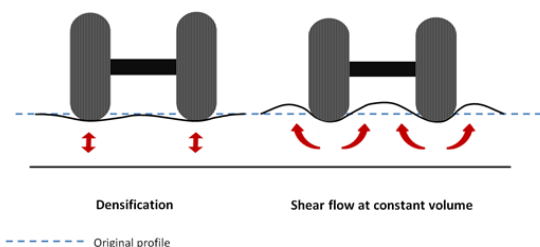


Figure 10: Rutting mechanisms

A typical development of rutting deformation is shown in Figure 11, with densification labelled as (1) and shear deformation labelled (2).

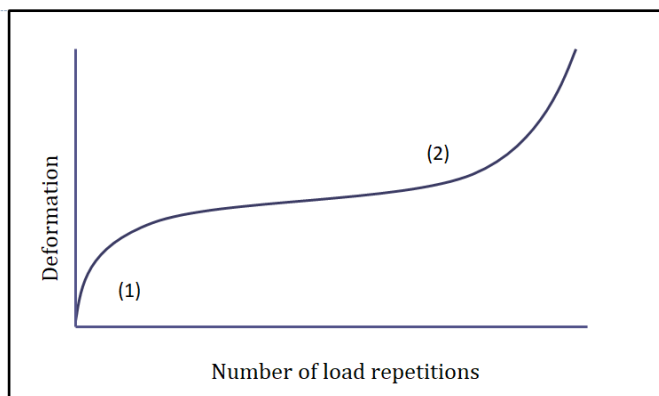


Figure 11: Rutting phases



#### Densification and volume decrease

During the initial densification a volume decrease takes place under action of traffic. Aggregates are pushed into a preferred orientation, leading to decrease in voids, typically from 7% to 4%. During this process of densification the mix stability improves.

Further densification may reduce voids to refusal density – typically to  $\approx 3\%$  although increased stability and binder ageing may counter this. In general mixes with voids decreasing to  $<3\%$  are more

prone to rutting than mixes with voids of 4%. Voids after primary densification are a measure of rut susceptibility of mix.

### Shear deformation

This phase of rutting occurs when the rut resistance afforded by the layer is overcome by the imposed stress state. The shear deformation is the summation of small flow displacements associated with repetitive traffic loads and gives rise to lateral movements causing a depression, or rut, to be formed.

Shear deformation is associated with movement or breakdown at particle-to-particle interface, afforded by:

- Cohesion afforded by the mastic (binder & filler)
- Macro-interlock by the aggregate skeleton
- Durability & frictional aspects of aggregate

Rutting is a frequently observed and serious type of distress in asphalt layers. This form of distress can lead to a significant reduction in road serviceability due to:

- Ponding of water in wheel tracks – a hazard in wet weather;
- Poor riding quality resulting in increased vehicle operating costs

South Africa has seen an increase in frequency in rutting on account of:

- Increased traffic volumes
- Increased axle loads and tyre pressures



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

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

Rutting can typically occur during prolonged summer pavement temperatures in excess of 45°C which frequently occur in South Africa. Under such conditions deformation is resisted primarily by the frictional resistance in the aggregate as binder resistance to viscous deformation and cohesion is somewhat compromised. Consequently the prevailing factor resisting permanent deformation would be associated with the mix type, e.g. stone or sand skeleton.

### External Factors

External factors affecting the extent and rate of rutting in asphalt are:



- Environmental conditions
- Material properties

### *Environmental factors*

Temperature is the most prominent environmental factor which affects the resistance of the binder to viscous flow and hence its shear resistance, especially above 45°C.

### *Material volumetric factors*

A volumetric aspect which is known to have a strong relationship with rutting performance is the percentage voids filled with bitumen (VFB). Overfilling of voids in the coarse aggregate with fine aggregates or overfilling voids in the fine aggregate with mastic should be avoided.

Aggregate characteristics that influence the packing characteristics of the aggregate are:

- shape, i.e. flaky, elongated, cubical, round
- surface texture, smooth or rough
- hardness and durability

## **Rutting evaluation**

As literature on the subject suggest that rutting performance is best correlated to wheel tracking tests, this type of testing has been selected in the design process, at the more advanced level. The laboratory wheel tracking test that has been evaluated and selected for inclusion is the *Hamburg Wheel Tracking Test* (HWTT). This test applies repeated tracking of a loaded steel wheel directly on an asphalt specimen, which is submerged in water at 50°C. The test therefore assesses the resistance of the asphalt to permanent deformation at an elevated temperature; it also assesses the resistance of the mix to separation of the binder and the aggregate (stripping) due to the presence of water.

At the more simplistic level of design (Level I) use is made of the Dynamic Creep Modulus to given an indication of the rut potential of the mix. In this test a cylindrical specimen is subjected to cyclical load at a prescribed temperature.

The use and application of these tests are covered in more detail in Section 5 – Laboratory Design Procedures.

## *Fatigue*



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.

Fatigue is a major consideration in in the design of asphalt and depends on the interaction of a number of elements:

- asphalt layer properties
- pavement structure
- the environment e.g. traffic temperature and moisture



The capacity of an asphalt layer to resist fatigue may be *worsened by stresses induced by thermal fluctuations* and loss of durability due to, for instance, binder ageing. High voids, which may accelerate binder ageing, or low binder content could lead to low fatigue life. Generally thin asphalt layers are more prone to fatigue as a result of high flexural deflections when compared with thick asphalt layers.



### Evaluation of fatigue resistance

Fatigue testing of asphalt mixtures involves subjecting asphalt mixture specimens to repeated loading.

The *Four-Point Beam Bending Test* is a standard test method for determining the fatigue life of compacted asphalt subjected to repeated flexural bending has been adopted in this method.

In the test, a beam specimen is subjected to a repeated, sinusoidal load, which is applied uniformly over the centre third of the beam. Usually, the failure of a test sample in a strain-controlled fatigue test is determined by the number of loading cycles at which a 50% reduction in initial stiffness is reached.

Further details regarding the testing of fatigue is covered in section 5.2.

### *Thermal fracture*



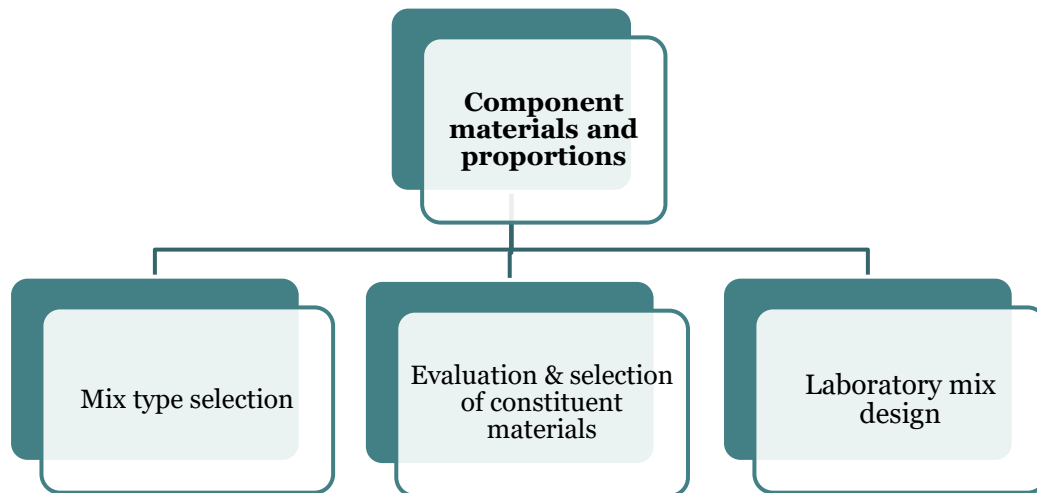
Thermal fracture of asphalt can arise due to contraction and expansion of the asphalt layer under extreme temperature changes. The potential for low temperature cracking is an interplay between the environment, the road structure and, importantly, the properties of the asphalt mixture, including the binder.



The performance grade specification provides criteria which will safeguard against the use of binders that are not unduly susceptible to thermal cracking. This will be achieved by limiting the stiffness of the binder, to avoid brittleness and setting requirements for the stress relaxation characteristics of the binder.

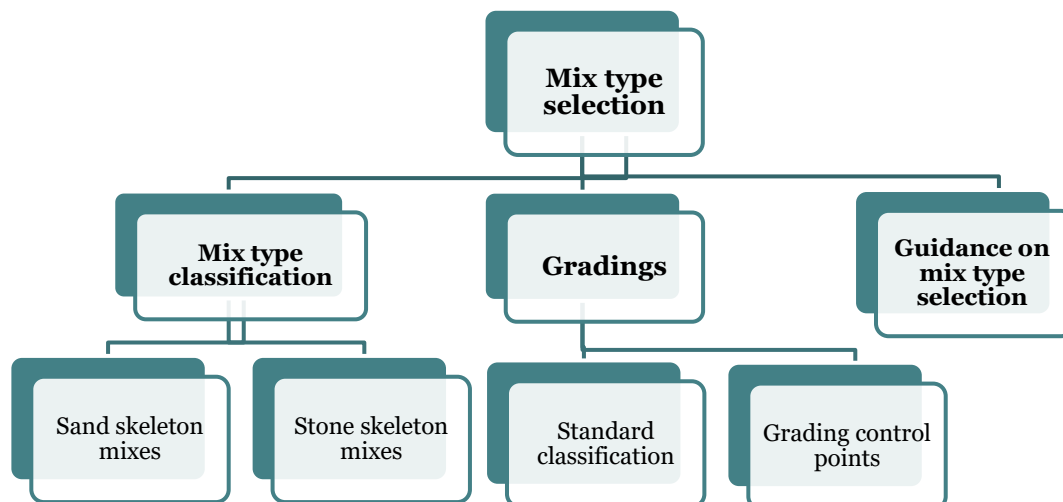
### 3. SELECTION OF CONSTITUENT MATERIALS AND PROPORTIONS

Having given consideration to the numerous factors that all have a bearing on the performance of the asphalt layer, this section will deal with how to optimise the composition of asphalt, in terms of constituent material types and their proportions to ensure that performance criteria are met. Specifically this section will cover mix type selection, evaluation and selection of constituent materials and the process of laboratory design.



#### 3.1 MIX TYPE SELECTION

This section will amplify the guidance given in Manual 35 regarding the selection of mix type as a point of departure in the mix design process as opposed to the selection of a grading, particularly in the design of mixes for new situations.





It goes without saying that the performance of an asphalt layer – its ability to counter the onset of damage in the form of rutting and fatigue – **is closely related to the structure of all aggregates in the mix.**

Not only is this concept confined to the mechanical properties related to load bearing; other important properties that contribute to the performance of the layer such as the permeability, durability and compactability are equally dependent on the manner in which the aggregate / filler / binder system is assembled.

The resistance to permanent deformation and the fatigue life of asphalt are improved by aggregate interlock. However, in striving for coarse aggregate interlock adequate workability or even permeability of the mix may be compromised.

**A clear understanding of the interaction between aggregate structure and mix performance is essential for the achievement of optimal mix proportions in HMA design.**



Consequently, Manual 35 recommends that this be the starting point of any **new mix** design process, rather than adopting a grading type or, worse, a grading envelope. The latter approach has the distinct disadvantage of having little or no evident bearing on the performance characteristics of the mix. For instance, mixes with the same (mass-based) grading could display significantly different behaviour depending on e.g. aggregate shape and surface texture.



Aggregate gradings are useful where mix types have been established and standardised for particular source materials and applications. Ultimately, gradings (together with aggregates shape and texture) are key to quality assurance procedures to ensure that mixes being laid on roads are representative of the materials used during the laboratory design process and can be expected to meet performance requirements.

In this context it is advisable to caution / deal with some specific factors related to aggregate packing.

### 3.1.1 MIX TYPE CLASSIFICATION



In Manual 35 mixes are 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.

### Sand-skeleton mixes



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

### Stone skeleton mixes



In stone-skeleton mixes the loads on the layer are carried by an interlocking matrix of the coarser aggregate fraction. Contact between the coarser aggregate fractions is achieved by ensuring that the finer aggregate fractions (mastic) do not overfill the air spaces available between the larger aggregate as will push the coarser aggregates apart.

To assure this, design methods for e.g. stone mastic asphalt (SMA) are particularly geared to reaching this aim. Achieving this situation results in the loads on the layer being carried predominantly by a matrix (or skeleton) of the coarser aggregate fraction. Examples include coarse continuously graded asphalt, stone mastic asphalt, ultra-thin friction courses, and open graded asphalt (porous) asphalt.

The aggregate configurations for sand- and stone-skeleton mix types are illustrated in Figure 12.

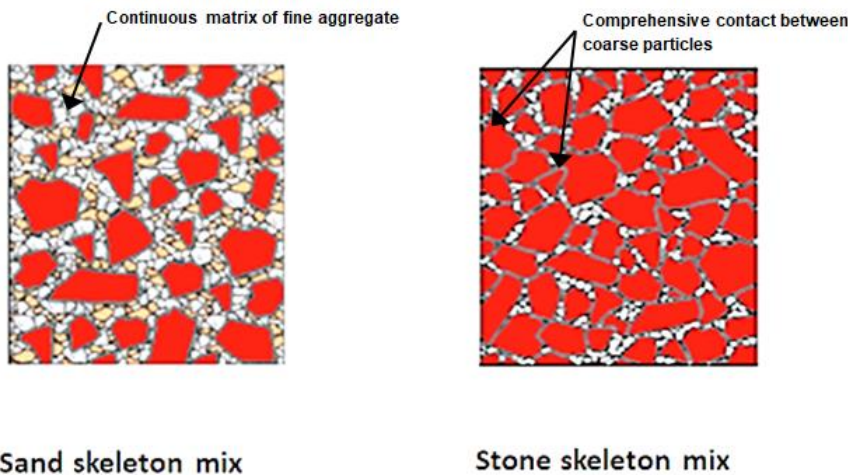


Figure 12: Mix types

### The Bailey method



There are a number of analytical methods to analyse and define aggregate structure. The method dealt with in Manual 35 is the Bailey Method (BM) to assist the designer in mix type selection and spatial / (volumetric) design.

TRB Circular  
E-C044



The (BM) presents a systematic technique to **establish which aggregate fraction – coarse or fine – is in control of the overall aggregate structure.**

What follows is a brief introduction to the principles involved in the BM and it is not the intention to deal with the BM in any further detail; it is covered in Sabita Manual 35 – Appendix A.

## Primary control sieve

A first principle of the BM is to define the **division between coarse and fine aggregate** of a specific mix. The determination of the respective volumes of these fractions leads to establishing which fraction is in control. The sieve size that defines the break between the coarse and fine fractions is termed the **primary control sieve (PCS)**. This size is the standard sieve closest to  $0.22 \times$  the NMPS. Table 2 shows the PCS size in relation to NMPS.

Table 2: PCS for various NMPS

NMPS (mm)	PCS (mm)
37.5	10
28	7.1
20	5
14	2
10	2
7.1	1

## Classification criteria

As a *general rule* the BM defines coarse graded (stone skeleton) and fine graded (sand skeleton) mixes as follows:

- Coarse graded:  $> 50\%$  of the aggregate blend is retained on the PCS (i.e.  $< 50\%$  passes the PCS)
- Fine graded:  $\leq 50\%$  of the aggregate blend is retained on the PCS (i.e.  $\geq 50\%$  passes the PCS).

Typical gradings of coarse- and fine graded aggregate blends are shown in Figure 13 and Figure 14, respectively.

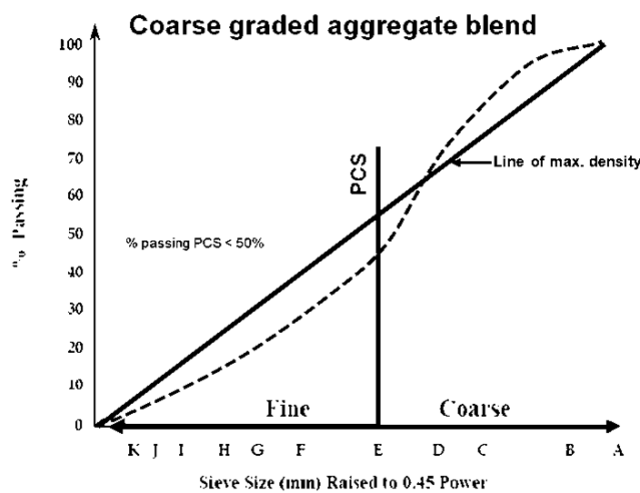


Figure 13: Coarse-graded blend

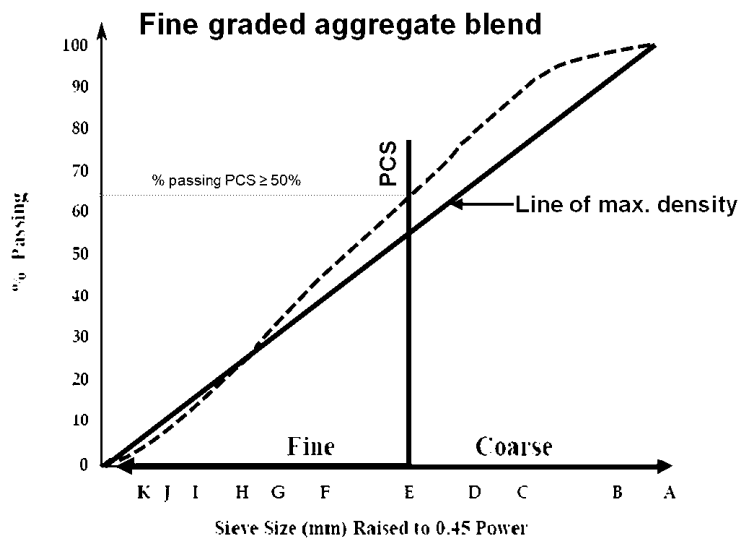


Figure 14: Fine-graded blend



To be more specific, standard tests (AASHTO T19) are performed to determine the *loose* and *rodded unit weights*, *LUW* and *RUW*, respectively of the coarse aggregate. From these the aggregate volume in a loose and rodded state can be determined which forms the principle basis of the determination of aggregate packing as follows:

- Sand skeleton mixes – the coarse aggregate volume (expressed as a *chosen unit weight, CUW*) is less than 80% of the volume corresponding to the *LUW*. In such cases the coarse aggregate “floats” (with little contact) in a matrix of fine aggregate which bears the load
- Stone skeleton mixes – the coarse aggregate volume is between 95 and 105% of the *LUW*. In such cases there is significant contact between the coarse aggregate particles which will largely bear the load.

It should be noted here that, in the case of SMA, aggregate volumes greater than that associated with 110 – 125% of the *RUW* is adopted to ensure considerable contact between the coarse aggregate particles.



**Note:**

**Mixes, especially continuously graded ones, with *CUW* between the boundary conditions listed above, i.e. between 80% of *LUW* and 95% of the *LUW*, may be problematic in terms of tenderness, a tendency to segregate, and compactability and should be avoided. Also the gradings of these mixes may lie close to the maximum density line which limits the simultaneous achievement of sufficient binder content AND air voids.**

Various gradings deriving from the various percentages of CA *LUW* are shown in Figure 15. Values of *CUW* below 80% of *LUW* represent fine graded (sand skeleton) mixes, whereas where this percentage is in excess of 90%, the mixes are coarse graded and will develop stone skeletons.



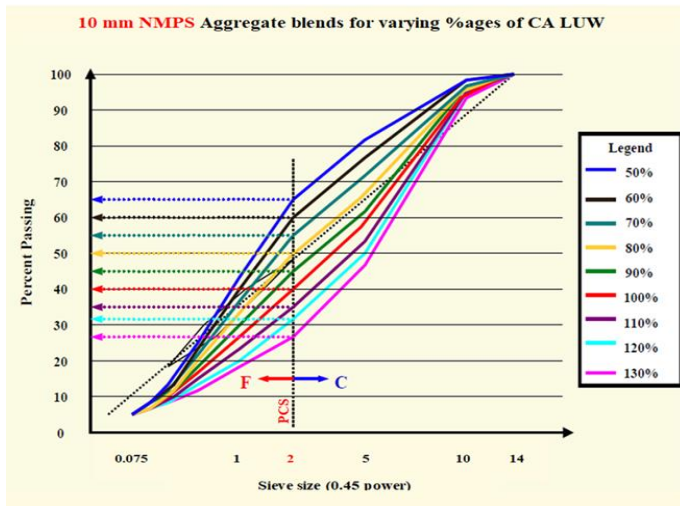


Figure 15: Aggregate grading for various % CA CUW

Source: The Heritage Group

**Note**

In the case of finely graded mixes i.e. sand-skeleton types, there is a further division of the fine aggregate, similar to that adopted for the entire mix. In this case a secondary control sieve i.e. the closet sieve to 0.22 x size of the PCS is determined and a similar volumetric process adopted.

**3.1.2 LIMITATIONS OF GRADING**



The designer should note that traditionally mixes were primarily classified in terms of their gradings. This practice has been discontinued in Manual 35, since traditional gradings that have been in use for decades do not necessarily guarantee optimal designs today. This is especially so in view of the change in aggregate shape over the years due to developments in crushing technology, and the increased heavy traffic loads that occur early in the life of the layer. Examination of the aggregate packing is now the primary step in the design of asphalt, to be followed by more complex testing and evaluation, to increase the confidence that the mix will perform as expected.

Some mix types, classified in terms of grading characteristics are shown in Figure 16. While a sand-skeleton mix type is quite distinct from stone-skeleton types such as “SMA” and “Porous”, it should be noted that continuously graded mixes could be either type, illustrating the limitations of using grading to characterise mix type behaviour and performance. Physical descriptions of the various grading types in general use are given in Appendix C.

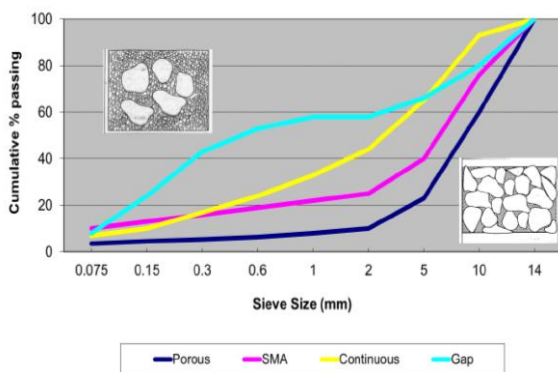


Figure 16: Grading types



Aggregate gradings have a crucial role in quality assurance. It is used to determine whether the aggregates from the approved source and parent material obtained from a mix sampled on a project have been mixed in the correct proportions, i.e. as reflected by mix proportions and grading.

### Standard classification

The generic classification of gradings in terms of aggregate packing is illustrated in Figure 17.

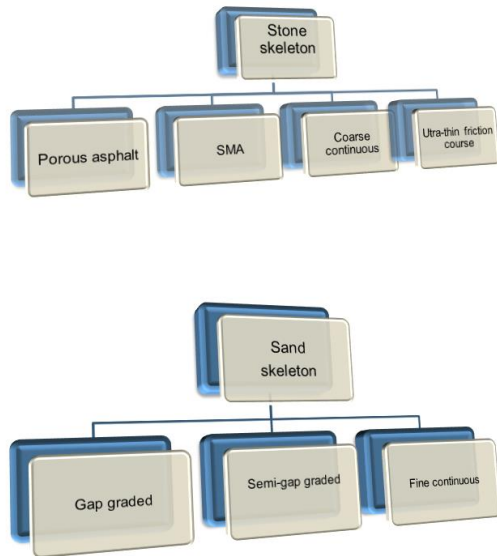


Figure 17: Mix type classification

### Grading control points

To achieve suitable aggregate packing to ensure that relevant performance characteristics of a mix are met, aggregates of various nominal sizes are mixed in pre-determined proportions. Such proportions are defined by the particle shape, texture and size distribution as represented by a grading.

The grading of aggregate is determined by a sieve analysis which is usually expressed in a table or in a graph. The analysis of grading in asphalt mix designs is generally limited to plotting the percent passing the sieves on a 0.45 power curve. Subsequent adjustments are based on moving this curve relative to a maximum density line. A brief explanation of the concepts behind this method of presentation of gradings is discussed in Appendix A Maximum density gradations.

To assist 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) as defined in section 0 should be selected in accordance with Table 1.
- The range of percentage passing the 2 mm sieve, and the 0.075 mm sieves as given in Table 12 of Manual 35.

Manual 35 provides an indication of typical NMPS for various applications (Table 3). **Nevertheless, these values of NMPS should still comply with the requirements of Table 1.** It also provides guidance on grading control points for four nominal maximum particle sizes of aggregates typically used for **production of sand skeleton (often continuously graded) asphalt mixes** in South Africa.

The control points for 20 mm NMPS are plotted in a 0.45 power chart in Figure 18 for illustration purposes.

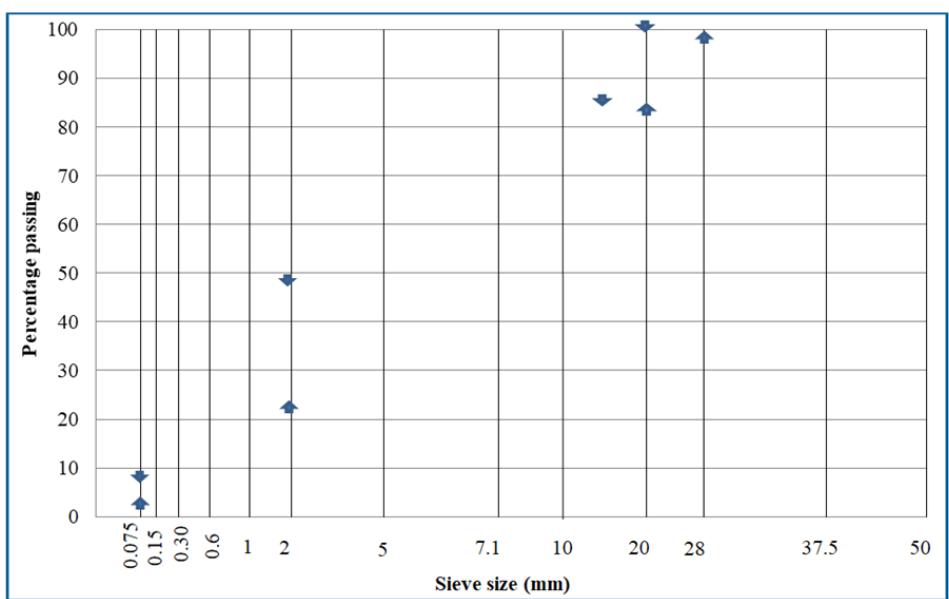


Figure 18: Grading control points for 20 mm NMPS

However the designer should note with care that:



1. These control points should be considered as **guidelines only** and are, at any rate, **not relevant to mixes such as stone skeleton types**



2. For stone skeleton mixes such as coarse continuous types and SMA it is strongly suggested that specific methods of aggregate proportioning, such as the Bailey method, need to be employed.

3.



The gradation of (continuously graded) sand skeleton mixes 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. (See discussion above in section 4.1.1.)



To optimise aggregate proportions, it is recommended that designers use the Bailey method<sup>3</sup>, which has been used with success in heavy duty asphalt applications in South Africa. In doing so, the designer should be mindful of the fact that some parameters of this method are based on aggregates encountered in the USA. Consequently, its application in South Africa should be approached with some caution. It should be noted, though, that the method provides valuable guidance to

<sup>3</sup> Published in Transportation Research Circular Number E-C044, October 2002

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.

### 3.1.3 GUIDANCE ON MIX TYPE SELECTION



It is emphasised here that the determination of aggregate packing characteristics of the mix (a stone-skeleton or a sand-skeleton type mix), is a critical choice to be made in the mix design process. Having taken into account all the relevant key elements of the design situation, as covered in section 4.3.1 above, an informed decision on the selection of the mix type can be made. In doing so, consideration should also 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 to moderate traffic applications, mainly in residential areas, are typically 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;
- The term “continuously graded asphalt” can encompass mixes with gradings varying from very coarse to very fine, for a particular maximum aggregate size; as such it has limited use.
- To ensure adequate skid resistance of gap-graded and semi gap-graded asphalt wearing courses, pre-coated chippings are usually spread on the freshly paved, hot mat prior to rolling

To guide the designer, Table 7 in Manual 35 rates a number of mix types manufactured with specific generic binder types, in terms of performance requirements of both wearing course and base layers. As a general guide, mixes with ratings of only 2 should be avoided where practicable.

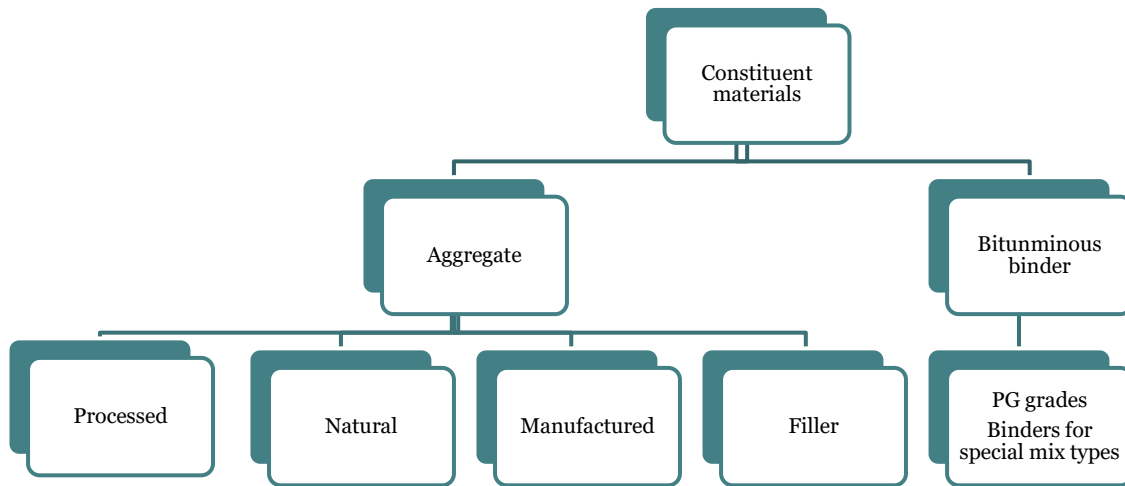


***Again it should be noted that, ultimately, terms such as A-E and A-R will be supersede by a PG binder classification framework terminology.***

In section 2.2 of Manual 35 factors that will influence mix type selection are listed as guidance.

## 3.2 EVALUATE AND SELECT CONSTITUENT MATERIALS

Manual 35 deals quite extensively with the process of evaluation and selection of constituent materials – stone aggregates and sand, filler material and bituminous binders. Stone aggregate, sand and fillers are dealt with under the heading of *aggregate materials*. It is the intention in this document to single out important aspects that the designer should keep in mind in the process of selection of the various mix constituents. Also, some aspects of quality assurance are covered.



In dealing with the selection of bituminous binders, this document will refer to the binder classification system adopted for the proposed PG specification for binders.

### 3.2.1 AGGREGATE

Aggregate materials typically constitute approximately 95% of the mass and 85% of the volume of asphalt mixes. The aggregate (also called the mineral aggregate) in asphalt primarily provides strength and bulk of the asphalt layer. The structural and functional performance of an asphalt mix in the pavement layer is greatly influenced by the physical properties and packing characteristics of the aggregate blend.

Aggregate sources / types are classified into four groups:

1. Processed aggregates
2. Natural aggregates
3. Manufactured aggregate (slag aggregates and reclaimed asphalt (RA))
4. Fillers

A few salient features of the above types are covered below.

#### *Processed aggregates*

This category covers aggregate that has been quarried, crushed and screened for further use. As these aggregates are generally mined by commercial quarry operations, one can expect the product to conform to general norms such as SANS 1083 and it would be reasonable to expect these to be of cubic and angular crushed shape suitable for asphalt mixes.



Particles that are flat, elongated, or both, should be avoided.

## Natural aggregate

This group comprises materials used in their natural form. They are mined from river beds, aeolian deposits and sometimes glacial deposits. The most commonly used natural aggregate for asphalt mixes is sand.



Whereas river deposits may be angular and afford a degree of interlock, aeolian deposits contain mostly single-sized, rounded particles which do not provide the interlocking matrix of river deposits. As such, while the use of aeolian deposits may promote workability their use may compromise the mix's shear stability (and thus susceptibility to rutting) which may have to be countered with measures to assure the layers resistance to permanent deformation. Such measures would typically comprise judicious selection of binder type and proportion of filler to stiffen up the mix.

The use of natural sands has often been the subject of critical appraisal, especially in the case of sand skeleton mix types such as gap and semi gap-graded mixes, for the reasons stated above. However, as good performance of mixes containing natural sand has been experienced in certain design situations it is recommended that their use should not be disregarded without investigation.

## Manufactured aggregate

This category comprises the by-products of industrial process, notably metallic slags or aggregates embedded in reclaimed asphalt (RA). The use of both these product types enhances sustainable practice and should always be investigated, especially the use of RA, which is widely available.

### Slag aggregates

The various types of steelmaking slags, i.e. blast furnace (BF) *iron slag* as well as basic oxygen furnace (BOF) and electric arc furnace (EAF) *steel slags* are covered in Manual 35, as is the use of ferro-chrome slag, listing precautions to be observed in their use in asphalt mixes.

In particular the attention of the designer is drawn to Note 4.1 in Manual 35 which states the following precaution for the use of steel slag:



***When 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.***



***Manganese slag should not be used as it poses a severe health risk.***

## Reclaimed asphalt

The reader is advised to consult Sabita Manual 36 / TRH21: *Use of reclaimed asphalt in the production of asphalt* on the use of this material and the design of mixes with RA. In this document RA is defined as follows:



Reclaimed asphalt (RA) is obtained from milling or excavation of existing bituminous pavement layers or from stockpiles of asphalt production overruns and returned material. The material so obtained is crushed and screened to ensure an acceptable maximum size and grading. Following the crushing and fractionating process, RA is stockpiled and loaded in such a manner as will enhance the uniformity of the material.



RA contains approximately 95% of high quality aggregate and 5% of aged bitumen, both valuable non-renewable resources. While the binder in the RA may have aged and hardened, the aggregate quality will not have altered and the RA should be treated as a valuable public asset.

In Manual 36 the proportion of RA in a mix is based on *RA binder replacement*<sup>4</sup> which will determine how the RA should be processed into fractions as well as the extent of testing of the material.

Sabita Manual 36 /  
TRH21



Important guidelines regarding the stockpiling of RA, as well as loading from these stockpiles, are given in Manual 36 and the designer should acquaint himself with these provisions, particularly with respect to quality assurance measures.

### Fillers

Fillers are defined as the material substantially passing the 0.075 mm (or 75 $\mu$ ) sieve and are essential components for producing asphalt mixes which are dense (where appropriate), cohesive, durable and resistant to damaging effects of moisture.

### Types

There are two types of filler:

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

Five kinds of filler in general use, as well as some characteristics and associated test methods and criteria, are given in Table 10 of Manual 35.

### Function

There is a wide range on opinions on the mechanisms whereby the performance characteristics of asphalt are affected by fillers, including:

- effects on the rheological properties of the binder / mastic system;
- simply a volumetric function to fill voids and bind aggregate particles together;
- improvement of the adhesion of binders and aggregates – particularly hydraulic fillers like lime; and
- improvement of compactability of the layer by using fly-ash.

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<sup>4</sup> The % age binder replacement in a mix,  $B_{RA} = \frac{B_{RA}}{B_T} \times 100$ , where  $B_T$  is the target binder content of the mix



It is not the intention to delve into this extensive field, rather it is suggested that ultimately the designer should aim to meet performance related properties of the mix as set out in the design procedures of Manual 35.

In addition to evaluating the properties of fillers, the designer should be mindful of:

- the relative costs of various fillers;
- their availability;
- storage potential; and
- relative effects on the stiffness of the binder.

Criteria that fillers should meet with regard to percentage passing 0.075 mm sieve size, as well as bulk density and void content are given in Chapter 4, Section 4.3.4 of Manual 35. Recommendations are also given for the chemical properties of fillers.



### Proportions

In determining the proportion of filler in the aggregate component of the mix, a fine balance must be struck. Sufficient filler should be added to ensure that the asphalt mix:

- has adequate cohesion, providing sufficient internal tensile strength and mix toughness to resist shearing forces (an important consideration for sand skeleton mixes);
- is durable; and
- has adequate resistance to permanent deformation.



Whereas this last aspect would be less important to stone-skeleton mixes, as the resistance to permanent deformation is mainly provided by stone-to-stone contact and aggregate interlock, adequate mastic viscosity would still need to be provided to prevent binder run-off to occur during the manufacturing, transport and placement of such mixes.

At the same time, one should guard against excessive proportions of filler that may increase the mix's susceptibility to brittle fracture as well as raising the viscosity of the hot mastic during the mixing and compaction process to such an extent that adequate compaction is not possible in the field. Conversely, a deficiency of filler tends to increase void content and may soften the mix, causing it to be tender during compaction and compromising resistance to rutting under traffic.

### Filler / binder ratio

A parameter used to define the proportion of filler in a mix is the filler / binder ratio.

In determining the filler / binder ratio of a mix the designer should take the following points into consideration:

- Tests carried out on a range of South African aggregates (the minus 0.075 fraction crusher dust) have shown that the binder-with-filler mastic may stiffen dramatically beyond a certain filler-binder ratio. It is recommended that the filler-binder ratio of wearing course mixes should not exceed 1.5, particularly for thin-layer mixes that cool more rapidly during paving and compaction. Because of their heat retention, higher filler-binder ratios can be allowed in thick asphalt bases (i.e. maximum ratio of approximately 1.6).





- While the bulk density of fillers may vary considerably, and hence the associated surface area, the filler / binder ratio used in design is a simple mass-based proportion:

$$\text{Filler / binder ratio (FBR)} = \frac{\text{mass of filler}}{\text{mass of binder}}$$



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



- Small increases in the amount of filler can literally absorb much of the binder resulting in a dry, unstable mix, and small decreases, i.e., too little filler will result in mixes which are excessively rich or “wet”.
- Consideration should be given to basing the filler / binder ratio on the effective binder content as this may have more relevance, as the binder absorbed into the aggregate would not have a role in the properties of the binder / mastic system. This is especially relevant where binder absorption is at the higher end of the typical range in dense mixes of 0.6– 1.2%. (Some client bodies may not allow the use of such highly absorptive aggregates.)



During construction, the tolerance limit of  $\pm 1\%$  of filler should be rigorously applied, except for open-graded mixes where the filler content could be as low as 2% filler and the  $\pm 1\%$  tolerance may be too high.

### 3.2.2 AGGREGATE STANDARDS

Processed, natural and manufactured aggregates are usually defined in terms of:

- type (geological or process origin)
- surface texture and absorptive properties
- spatial dimensions – shape and size
- “strength” as defined by:
  - hardness;
  - toughness;
  - soundness;
  - durability;
- uniformity of physical properties
- cleanliness

At the time of compiling this document SANS 1083 is under revision and will now cover ALL aggregates, sands, gravels and secondary materials utilised for road construction. It is not the intention to repeat these standards here. As far as the selection and tests on aggregates for asphalt are concerned requirements are listed in Table 16 of Manual 35.

SANS 1083



Sabita Manual 35



It should, however, be mentioned that the proposed COTO specifications currently being compiled also limit the extent of binder absorption – a maximum of 0.5% by mass.

## Uniformity

Aggregates are normally acquired in screened fractions of nominal size with grading control limits assigned to the specific fraction. In the design process blending proportions of these “single sized” aggregates are determined to comply with the requirements set in terms of the desired aggregate packing configuration which, in turn, will contribute to meeting the desired performance characteristics.

The revised SANS 1083, mentioned above, provides for three classes of aggregate in terms of their uniformity and quality. For both coarse and fine aggregates, Class 1 is generally prescribed for stone skeleton mixes, whereas Class 2 is recommended for sand skeleton mixes.



It is important to note that, in cases where aggregate fractions readily and economically available from a particular source may not comply strictly with the “single size” grading requirements, the aggregates may be eminently usable to achieve acceptable blends, **PROVIDED THAT THE QUALITY OF THE DELIVERED AGGREGATE IN TERMS OF SHAPE SIZE AND TEXTURE ARE CONSISTENTLY MET.**

If uniformity of aggregate cannot be achieved, it is advisable to investigate alternative sources; otherwise, designs should be prepared to cope with the variability, which may quite a challenging goal to be considered by experts.



While the relaxation of standards for nominal size fractions may be acceptable in the general case, in the case of high stone content mixes like SMA, where skeletal structure is of critical importance, relaxations on compliance with grading envelopes will not be good practice.

The designer should always be mindful of the fact that aggregate grading alone will not be a determining factor; other factors such as particle shape and surface texture play an important role in the mix's performance characteristics.

## Filler

The required standards for filler are defined in both physical and chemical terms.

### Physical

In terms of physical properties fillers should meet the following criteria:

- Percentage mass passing the 0.075 mm sieve: minimum 70
- Bulk density in toluene: 0.5 – 0.9 g/mℓ
- Voids in the compacted filler: 0.3 – 0.5%

### Chemical



Fillers from natural sources that have excess clay minerals or adsorption potential will have adverse effects on the mix in terms of premature hardening and stripping. To avoid such deleterious materials it is recommended that the Methylene Blue test is used as a means of assessing the amount and activeness of clay minerals in the filler.

The test is carried out by dispersing a 1 g sample of material passing 0.075 mm in water. This is titrated with an indicator solution made by dissolving methylene blue in distilled water. The indicator solution is gradually added to the dispersion. After agitation, a drop of the dispersion is removed

using a glass rod and dabbed onto a sheet of filter paper to form a blue spot. The indicator solution is added in increments of 0.5 mL. The dabbing procedure is repeated after each increment of the indicator until a definite blue halo appears around the spot. The quantity of methylene blue used to achieve the halo effect is recorded and used to calculate the methylene blue adsorption value (MBV).

Experience has shown that methylene blue values of 5 or less are indicative of high quality filler. Fillers with methylene blue values above 5 should be further investigated by means of hydrometer analysis and the determination of Atterberg limits.

### 3.2.3 BITUMINOUS BINDER

#### *Introduction of a PG specification*

In Manual 35, the use of a performance grade (PG) system of binder classification and compliance requirements is introduced, since the implementation of this system as a basis for standard specifications is imminent. It is not the intention to cover this complex topic extensively here; rather the reader is referred to the Sabita publication *Technical Guideline:*

Sabita Technical  
Guideline on PG  
binders



*The Introduction of a Performance Grade Specification for Bituminous Binders February 2017.*

The term “performance grade” refers to the concept that binders will be graded in terms of their use and application to perform in specific environments defined by climate (particularly temperature ranges) and the intensity *and rate* of traffic loading.



Ultimately it is foreseen that the PG specification published by SABS as SATS 3028 will replace the national standard for bitumen – SANS 4001-BT1, as well as the test parameters and compliance limits recommended in TG1: *Technical Guideline: The Use of Modified Bituminous Binder in Road Construction.*

The PG specification is based on engineering properties of bituminous binders to ensure optimal performance of bituminous layers, given a set of operating circumstances. Additionally, is binder-blind, i.e. it does not prescribe or differentiate between neat binders and those whose properties have been modified by the addition of e.g. a polymer substance. The need for modification and the type of modification rests with the binder suppliers.

The PG specification also covers criteria for *safety and ease of handling.*

#### *Performance requirements*



The key feature of this new specification is limiting the potential of the binder to contribute to:

- permanent deformation
- fatigue cracking and
- low temperature fracture.

In addition they should be durable i.e. maintain their properties to counter distress for a long period.

The designer should note **that other factors e.g. aggregate composition, binder film configurations all contribute to the behaviour of the mix.** While all these aspects are dealt with in the method, the provisions of the PG specification for binders underpin the achievement of design objectives.

As a result, the concept of damage resistance characteristics (DRC) was introduced to provide a specification framework to gauge the binder’s resistance to damage resulting from:



- Permanent deformation (viscous flow) – at elevated temperatures and slow rates of loading;
- Cracking – at intermediate temperatures; and
- Fracture – at low temperatures

### Performance tests



Bitumen exhibits visco-elastic behaviour. In short, this means that at high temperature and low rates of loading associated with slow moving traffic (such as at intersections and crawler lanes) bitumen behaves much like a viscous solid and is prone to permanent deformation (viscous flow). At low temperatures and higher loading rates associated with fast moving traffic bitumen behaves more like an elastic solid and may recover nearly fully from an imposed deformation.

The science that describes this kind of behaviour is termed *rheology* which is the study of the flow and deformation of matter, including soft solids under conditions in which they flow rather than recovering fully after deformation.

This visco-elastic behaviour of bitumen is presented schematically in Figure 19.

*Rheometry* refers to the laboratory measurement techniques to determine the rheological properties of materials. For bituminous binders this entails the measurement of both elastic and viscous behaviour under diverse conditions, primarily the type and frequency of loading and temperature of the specimen being tested.

Two types of instruments in general use globally will be covered in this document:

- Dynamic shear rheometer (DSR)
- Bending beam rheometer (BBR)

Sabita Technical  
Guideline on PG  
binders



SATS 3028

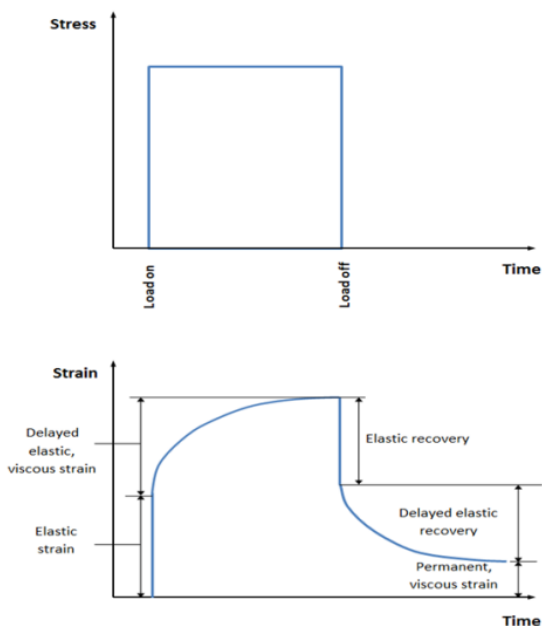


Figure 19: Response of bitumen in a simple creep test



Briefly, the tests undertaken to characterise bitumen visco-elastic behaviour are as follows:

- Resistance to viscous flow: Non-recoverable creep compliance,  $J_{NR}$  using the DSR
- Resistance to low temperature / non-load associated fracture: Measuring stiffness ( $S$ ) and stress-relaxation characteristic ( $m$ ) with the BBR
- Resistance to fatigue and durability cracking is assessed by determination of an entity  $\Delta T_C$  which reflects the difference in temperatures at which the stiffness ( $S$ ) has a maximum value and the stress relaxation factor ( $m$ ) is a minimum value.
- In these tests the binder is subjected to both short term and medium term ageing, using the rolling thin film oven (RTFO) and the pressure ageing vessel (PAV)

### *Specification framework*

From a *performance* perspective the specification framework therefore provides a rational framework for the selection of binders on the basis of:

- the operating environment i.e.:
  - climate (max and min temperatures)
  - traffic volumes and speed
- resistance to viscous flow
- adequate durability against fatigue; and
- low temperature fracture

In the light of the above considerations the specification framework provides for requirements categorised in terms of climate and traffic loading.

### **Climate**

The effect of climate is taken into account by a grading designation component related to:

- the average seven-day maximum pavement design temperature, and
- minimum pavement design temperature.

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



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

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

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

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.  $[(T_{max} + T_{min})/2 + 4]^{\circ}C$
- **Low temperature:  $T_{min}$** : 10°C above the minimum grading temperature,  $T_{min}$ , i.e.  $[T_{min} + 10]^{\circ}C$

The maximum temperature appropriate for the selection of the binder for a particular project can be determined from maps given in Manual 35.

### Traffic

Traffic will be classified in terms of *Standard, Heavy, Very Heavy* and *Extreme* categories with the associated symbols of S, H, V and E, respectively. **The classification is based on both traffic volume and traffic speed.**

For convenience, *Table 8* of Manual 35 is reproduced as Table 3 below as guidance. As can be seen, the design situation is more severe for low operating speeds. Hence, a particular design level may have to cater for a range of severity in terms of the combined effect of traffic volume and speed. In cases of traffic categories H, V & E the advice of an experienced designer should be sought. For reference, the mix design level (of sophistication) adopted for the various levels of traffic, is shown in the last column.



The designer should note that these design levels (IA – III) relate to the traffic loads and volumes (expressed as E80s) **over the service life of the asphalt pavement.**

The intention is to adopt an appropriately advanced design procedures to mitigate exposure to the risks associated with structural damage.

**Table 3: Binder grade selection on the basis of traffic speed and volume**

Design traffic (million E80)	Mix design level for given traffic operating speed (km/h)			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	

Traffic loading conditions

- S – Standard
- H – Heavy
- V – Very Heavy
- S – Severe
- E – Extreme

The binder classification system adopted for SA is represented in Table 4.

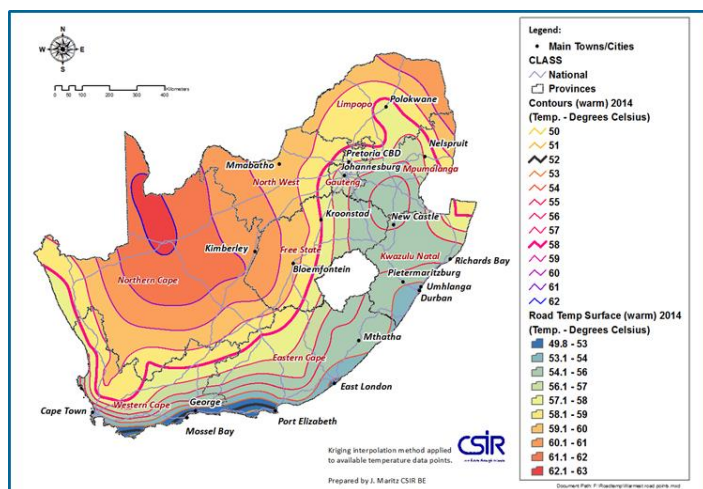
**Table 4: Binder classification**

Classification	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 $T_{mn}$ (°C)	-22				-16				-10			

*Selection of binder grade*

As indicated in Sabita Manual 35, the 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 of the manual (reproduced in Figure 20 below) indicating the 7-day average maximum asphalt temperatures at 20 mm depth in the asphalt.
2. If the asphalt layer is to be located wholly or partially within the > 58°C Zone, a PG 64 binder is selected; or
3. If the asphalt layer is to be located wholly within the ≤ 58°C Zone, a PG 58 is selected (a PG 64 will also conform to minimum requirements)
4. Determine the traffic level and average speed and choose the correct grade of binder as indicated in Table 3.



**Figure 20: Average 7-day maximum temperature at 20 mm depth in asphalt layer**

*Notes on the selection of binder grade*



It is recommended that the designer, in considering the PG grade of binder to be used in a specific application, take into account the following general factors:

- In essence the PG specification is a purchasing specification, i.e. it facilitates the optimal selection of a binder for a particular project in terms of the operating environment (traffic and climate) to achieve specific performance requirements.
- However, having selected an appropriate grade of binder does not in itself assure adequate performance of an asphalt layer. In the asphalt layer the binder films operate in conjunction

with the spatial arrangement of aggregate (including filler) to meet performance requirements and load distribution to the substrate. For instance, a high grade, hard binder alone may not in itself be capable of resisting viscous flow (plastic deformation) of the layer. The aggregate configuration (e.g. a stone skeleton) will almost certainly be required to resist this type of distress. Similarly limiting the ingress of air and moisture to protect the asphalt against premature ageing, would be jointly countered by having sufficiently thick films of a durable binder as well as an aggregate structure (sand skeleton) to disperse void spaces. In instances where a very stiff asphalt base layer is required to distribute loads to the substrate, the viscous properties of the binder will play a predominant role, especially during periods of high temperature.

- The PG specification does not differentiate between straight and modified bitumen. Whether to modify or not is a decision for the binder manufacturer in order to enhance the rheological properties of the neat binder to meet a specific grade of binder. In all probability the 58S and 64S grades (refer Table 4) would be straight run bitumen, whereas all the other grades may be specially produced bitumen or polymer-modified. Therefore the designer is in a favourable position of simply selecting a *performance* grade without having to consider arbitrary classifications defining e.g. elastomers or plastomers.

### Binder selection for specific mix types



Until such time when a performance grade specification is fully implemented, binder selection would be based on the current specification - SANS 4001-BT1 - and guidelines in AsAc TG1. Manual 35 gives specific recommendations on the interim procedures for selecting binder for specific asphalt types. These are shown in Table 5.

Table 5: Interim binder selection guide for various asphalt types

Asphalt Type	Reference Document
“Enrobé à Module Élevé” or EME	Sabita Manual 33: <i>Design of High Modulus Asphalt (EME)</i>
Sand asphalt	Sabita Manual 18: <i>Appropriate standards for the use of sand asphalt</i>
Asphalt for lightly trafficked roads in residential areas	Sabita Manual 27: <i>Guidelines for thin hot mix asphalt wearing courses on residential streets</i>
Porous asphalt mixes	Sabita Manual 17 <i>Porous asphalt mixes: Design and use</i>
Bitumen rubber asphalt	Sabita Manual 19 <i>Guidelines for the design, manufacture and construction of bitumen rubber asphalt wearing courses.</i>
Warm mix asphalt	Sabita Manual 32: <i>Best practice guideline and specification for warm mix asphalt.</i>
Reclaimed asphalt binder	Sabita Manual 36 / TRH 21: <i>Use of reclaimed asphalt in the production of asphalt.</i>



## 4. LABORATORY DESIGN PROCEDURES

Having gone through the process of evaluating the design situation, establishing design objectives and consideration of available component materials, the designer is now in a position to proceed to the laboratory design phase. (See Figure 21)

The process is presented in detail in Manual 35; the objective here is to draw the attention of the designer to specific, important aspects that would be encountered during the design of an optimum mixture for a specific application.

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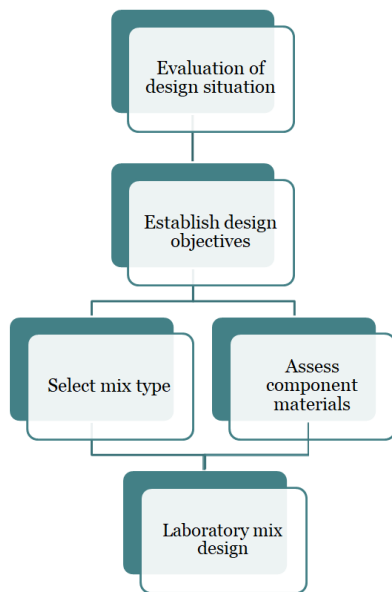
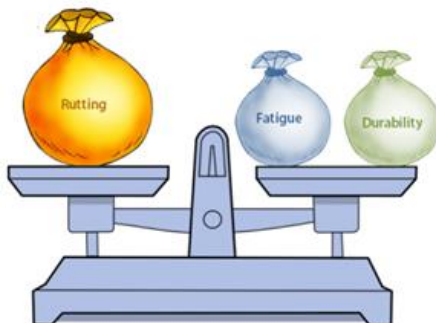


Figure 21: Laboratory mix design in relation to preceding sections

### 4.1 MIX DESIGN OBJECTIVES

Manual 35 gives a clear statement on the objectives of this phase.





The designer should be aware that the design of asphalt **often entails meeting conflicting requirements** as regards the satisfactory performance of an asphalt layer. For instance, measures to counter permanent deformation (e.g. rutting) on the one hand and adequate fatigue resistance and durability, may conflict with one another and would require careful consideration to find the optimum composition of the mix whereby all potential forms of distress can be countered.

The art of asphalt mix design, therefore, inevitably requires the ability to strike a balance whereby the expected performance of the mix is optimised through the judicious selection of component materials and their proportions. Table 6<sup>5</sup> below illustrates a number of conflicting design measures that would be considered to meeting e.g. requirements in respect of rutting, fatigue and durability.

**Table 6: Conflicting design aims**

Design objective	
Rut resistance	Fatigue strength / durability
Design considerations	
Reduced binder content with due consideration of binder film thickness	Increased binder content
Highly viscous / non-compliant binder	Binder with stress relaxation properties
Increase void content	Reduced void content
Stone skeleton mixes	Sand skeleton mixes
High filler / binder ratio	Reduced filler / binder ratio

## 4.2 DESIGN PROCEDURE

A novel feature of Manual 35 is presentation of a design framework that provides for four levels of complexity of mix design.



This approach provides for the selection of a **design procedure that is appropriate for the design situation** in terms of the importance of the road. For low and / or light traffic on access roads, parking lots and driveways the design procedure should be fairly straightforward and simple as the consequence of a design inexactness is not nearly as severe on this type of facility as it would be on, say, a major, heavily trafficked urban free. In the latter case, a design deficiency can have major disruptive and safety consequences. Hence for such situations the adoption of a more advanced and precise design procedure is justified to mitigate exposure to the risk of premature structural damage.

The designated levels are:

- Level I A & B
- Level II, and
- Level III.

<sup>5</sup> This table is not comprehensive and merely serves to illustrate the principle.

These are associated with traffic intensity expressed as the number of repetitions of 80 kN axles over the design period (expressed as E80s) **over the service life of the asphalt pavement** as shown in Table 7.



It needs to be stressed that the service life of a pavement may be 20 years or more. The expected cumulative traffic over this period determines the design level adopted, **NOT** the traffic over the expected life of the asphalt layer.

**Table 7: Mix design levels for traffic volumes over the service life of the pavement**

Design traffic [E80]	Mix design level
< 0.3 million	Level IA
0.3 to 3 million	Level IB
>3 to 30 million	Level II
> 30 - 100million	Level III
> 100 million	

### 4.3 VOLUMETRIC DESIGN PROCESS



Volumetric design is the starting point for ALL LEVELS of design. If correctly carried out it provides the designer with a mix that has a balanced composition and a foundation for further testing and, possibly, fine-tuning that will assess the capacity of the mix to meet certain performance requirements.

A number of standard volumetric concepts are used in the design of asphalt mixes. These are often determined in standard tests or formulas based on such test results. The concepts used in SA are set out below.

#### 4.3.1 TEST STANDARDS USED IN SA IN MIX DESIGN PROCEDURES



Sabita Manual 39 Protocol ASP 8: *Protocol for Determining the Bulk Density of Compacted Asphalt Mixtures* is used for the determination of bulk density and void content of compacted asphalt and Protocol ASP9: *Protocol for Determining The Maximum Voidless Density of Asphalt Mixtures* of that Manual for the determination of the maximum void-less density of asphalt mixes and the quantity of binder absorbed by the aggregate.

The nomenclature for volumetric parameters used in the SANS standard is summarised Table 8.

Table 8: Density parameters used in volumetric analysis

Parameter	Description	Unit	Test standard
<b>BD<sub>A</sub></b>	Dry Bulk Density of aggregate retained on the 5 mm sieve	kg/m <sup>3</sup>	3001-AS20
	Dry Bulk Density of aggregate passing the 5 mm sieve	kg/m <sup>3</sup>	3001-AS21
<b>AD<sub>A</sub></b>	Apparent Density of material retained on the 5 mm sieve	kg/m <sup>3</sup>	3001-AS20
	Apparent Density of material passing the 5 mm sieve	kg/m <sup>3</sup>	3001-AS21
<b>W<sub>ABS</sub></b>	Water absorption of material retained on the 5 mm sieve	kg/m <sup>3</sup>	3001-AS20
	Water absorption of material passing the 5 mm sieve	kg/m <sup>3</sup>	3001-AS21
<b>BD<sub>B</sub></b>	Bulk density of binder (1020 kg/m <sup>3</sup> default) <sup>6</sup>	kg/m <sup>3</sup>	ASTM D70
<b>BD<sub>MIX</sub></b>	Bulk density of the mix for mixes with VIM > 12%	kg/m <sup>3</sup>	Sabita Manual 39: ASP8
	Bulk density of the mix for all other mixes	kg/m <sup>3</sup>	AASHTO T331
<b>MVD</b>	Mass per unit volume of a void-less bituminous mixture at a known test temperature	kg/m <sup>3</sup>	Sabita Manual 39: ASP9

<sup>6</sup> 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

### 4.3.2 MASS AND VOLUME CONCEPTS

The various mass and volume concepts are illustrated in Figure 22.

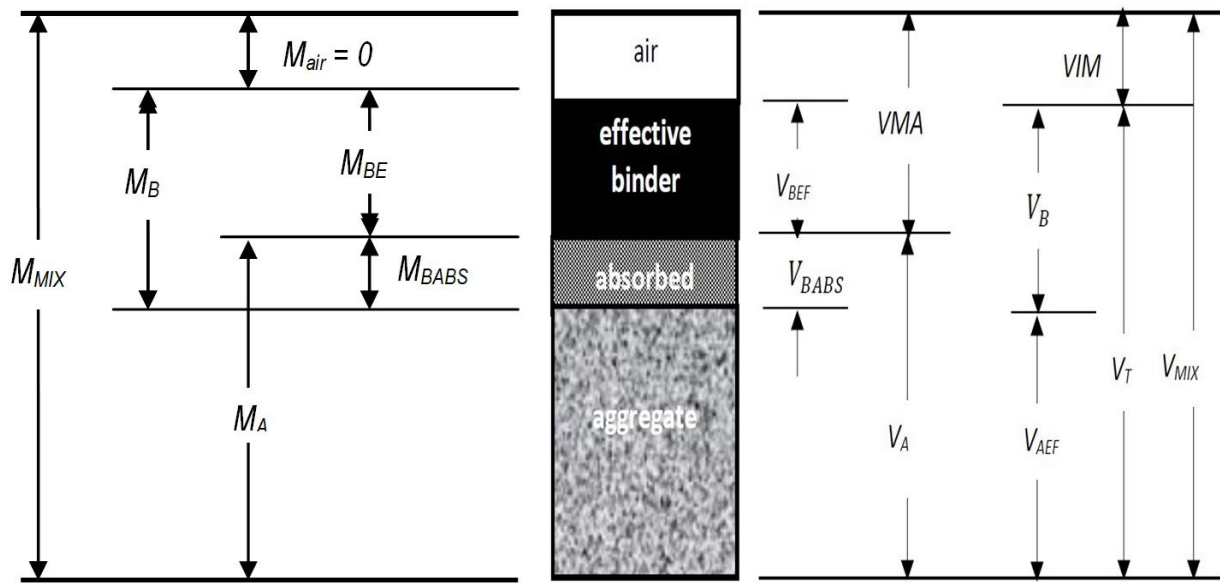


Figure 22; Definition of mass and volumes of mix components

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
$M_{BE}$	Mass of effective binder	$V_{BEf}$	Volume of effective binder
$M_A$	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

### 4.3.3 DETERMINATION OF MIX DESIGN PARAMETERS

The determination of the parameters used in the design of asphalt is presented below. For further details see ASP8 and ASP9 of Sabita Manual 39.

#### Bulk Density of the Mix



For both design purposes and quality control purposes the volumes associated with bulk density of asphalt is determined either by the vacuum sealing (VS) method (AASHTO T 331) or by measurement of dimensions. For dense mixtures, other than those purposely designed to be porous (i.e. with void content in excess of 12%) –the VS method is used.

For mixtures purposely designed to have an open surface and high porosity, with void content > 12 %, by direct dimensional measurement

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

$$BD_{MIX} = Gmb\rho_W \quad (1)$$

where:

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

$\rho_W$  = density of water at 25°C, ( $997.1 \text{ kg/m}^3$ ).

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

$$V_{MIX} = l \times b \times h \quad (2a)$$

or

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

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 ( $\text{kg/m}^3$ ) is calculated as:

$$BD_{MIX} = \frac{10^6 \times M_4}{V_{MIX}} \quad (3)$$

where

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

### **Maximum Voidless Density of the Mix**

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

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

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

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

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

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

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

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

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

$$MVD = \left\{ \frac{(M_2 - M_1)}{\frac{((M_4 - M_1) - (M_3 - M_2))}{\rho_w}} \right\} \quad (5a)$$

(b) When water is absorbed by the coated aggregate<sup>7</sup>:

$$MVD = \left\{ \frac{(M_2 - M_1)}{\frac{(M_4 - M_3 + M_5)}{\rho_w}} \right\} \quad (5b)$$

### **Additional volumetric calculations**

The mass of binder in the mix, expressed in grams (g), is:

$$M_B = B \left( \frac{M_2 - M_1}{100} \right) \quad (6)$$

Where:

$M_B$  is the mass of binder in the mix, expressed in grams (g);

$B$  is the is the percentage binder in the mix, and is expressed as a percentage of the mass of the mix (aggregate plus binder)

The mass of the aggregate in the mix, expressed in grams (g), is:

$$M_A = \frac{(100 - B)(M_2 - M_1)}{100} \quad (7)$$

The volume of binder in the mix, expressed in cubic centimetres (cm<sup>3</sup>) is:

$$V_B = \frac{1000M_B}{BD_B} \quad (8)$$

where

$BD_B$  is the bulk density of the binder, expressed in kilograms per cubic metre (kg/m<sup>3</sup>).

The volume of aggregate in the mix, expressed in cubic centimetres (cm<sup>3</sup>), is:

$$V_A = \frac{1000M_A}{BD_A} \quad (9)$$

where

$BD_A$  is the bulk density of the aggregate, expressed in kilograms per cubic metre (kg/m<sup>3</sup>).

The total volume of aggregate and binder in the mix, expressed in cubic centimetres (cm<sup>3</sup>), is:

$$V_T = V_A + V_B - V_{BABS} \quad (10)$$

The de-aired volume of the mix, expressed in cubic centimetres (cm<sup>3</sup>) is:

$$V_{DA} = \frac{1000}{\rho_w} (M_4 + M_3 - M_5) \quad (11)$$

The volume of binder absorbed, expressed in cubic centimetres (cm<sup>3</sup>), is:

---

<sup>7</sup>) Cases where 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 6.3 of SANS 3001-AS11.

$$V_{BABS} = V_A + V_B - V_{DA} \quad (12)$$

The *mass of binder absorbed in the mix*, expressed in grams (g) is:

$$M_{BABS} = \frac{(V_{BABS} \times BD_B)}{1000} \quad (13)$$

The *percentage binder absorbed by the aggregate in the mix*, expressed as a percentage of the mass of the dry aggregate in the mix, is:

$$B_{ABS} = 100 \left( \frac{M_{BABS}}{M_A} \right) \quad (14)$$

The *mass of effective binder in the mix*, expressed in grams (g) is:

$$M_{BEF} = M_B - M_{BABS} \quad (15)$$

The *percentage of effective binder* in the mix, expressed as a percentage of the mass of the mix (aggregate plus binder), is:

$$B_{EF} = \frac{100M_{BEF}}{M_A + M_B} \quad (16)$$

The *percentage Voids in the Mix (VIM)* is the difference between the maximum void-less density (*MVD*) and  $BD_{MIX}$ , expressed as a percentage of *MVD*:

$$VIM = 100 \left( \frac{MVD - BD_{MIX}}{MVD} \right) \quad (17)$$

The *volume of voids in the mineral aggregate* (without binder) in the compacted mix, *VMA*, expressed as the percentage difference between the volume of the aggregate in the mix and the bulk volume of the mix:

$$VMA = 100 - \left( \frac{(100 - B)BD_{MIX}}{BD_A} \right) \quad (18a)$$

Alternatively, if the percentage voids in the mix, *VIM*, and the volume of effective binder expressed as a percentage of the volume of the bulk mix,  $V_{BEF}$ , are known, *VMA* can be calculated as:

$$VMA = VIM + V_{BEF} \quad (18b)$$

The *volume of effective binder*,  $V_{BEF}$ , expressed as a percentage of the volume of the bulk mix, is:

$$V_{BEF} = \frac{B_{EF}BD_{MIX}}{BD_B} \quad (19a)$$

Alternatively, if *VMA* and *VIM* are already known,  $V_{BEF}$  can be calculated as:

$$V_{BEF} = VMA - VIM \quad (19b)$$

The *percentage of voids in the compacted mix filled with effective binder*, *VFB*, expressed as a percentage of the volume of voids in the bulk mix, is:

$$VFB = \frac{100V_{BEF}}{VMA} \quad (20)$$

### **Notes on density measurements on aggregate**

The three generally accepted types of density for aggregate use in asphalt are the following:



- Apparent density ( $AD_A$ );
- 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<sup>8</sup> soaking. It does however include pores that are not accessible to water penetration.

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

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

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

These volumetric concepts (denominators in density expressions) related to aggregates are illustrated conceptually in Figure 23. The volumes associated with the determination of densities listed are indicated with blue outlines.

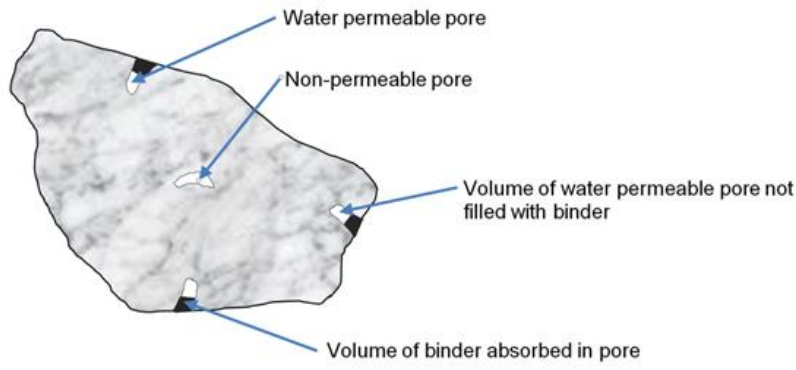
Since  $AD_A$ ,  $BDA$  and  $BDAEF$  all have the same mass as numerator of the density expression and only differ in the volume as denominator, the following *inequality* is always true.

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

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

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<sup>8</sup> 15 – 19 hour duration



Bulk Density  $BD_A$



Apparent Density  $AD_A$



Effective Density  $BD_{AEF}$

Blue lines schematically depict the boundaries of the relevant volume used for the particular density parameter.

Figure 23: *Volumes* associated with the determination of Bulk, Apparent and Effective Density of Aggregate

#### 4.3.4 MIX COMPOSITION

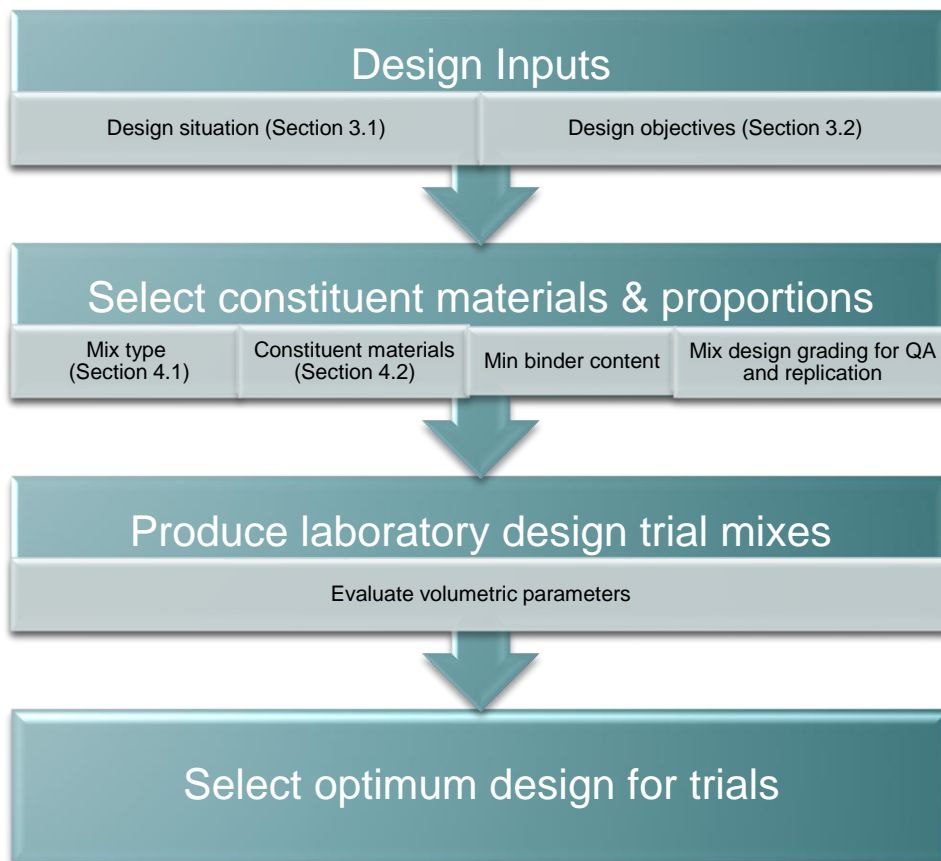


Figure 24: Design framework

The process adopted for selecting the constituent materials of the mix, within the context of the whole process, is depicted in Figure 24. Whereas the selection of mix type and constituent materials have been covered in section 3, the question of establishing a minimum binder content will be elaborated on here.

#### 4.3.5 MINIMUM BINDER CONTENT



It has been mentioned a number of times that having a sufficient volume of bituminous binder in the mix is a critically important volumetric aspect. This will ensure that aggregate (including filler) has a adequately thick film of bitumen coating to provide cohesive strength to render the asphalt sufficiently flexible, resilient and, especially, durable.

While the binder content of a mix is generally expressed as a **mass based** proportion (or %), this measurement does not necessarily reflect the aspect of sufficient **volume**. This is because the bulk density of various aggregates types used in similar proportions (gradings) may vary significantly, giving rise to substantially different binder volume concentrations for a given binder proportion by mass. For example, whereas the bulk density of quartzite and dolomite is in the region of 2.65, dolerite basalt and andesite have bulk densities in the region of 2.8. A binder content (m/m) of, say, 5.8% for a quartzitic sandstone aggregate blend would be equivalent to a 5.5% (m/m) binder content for a basalt/dolerite/andesite to achieve the same binder volume for coating aggregates.

## Binder film thickness

To deal with this important aspect the concept of a minimum binder film thickness requirement is adopted in Manual 35 to ensure that:

1. A binder content that will provide sufficient coating of aggregates is adopted; and
2. That the bulk density of the actual aggregate blend is taken into account to adjust the binder concentration accordingly.

The determination of the minimum binder content entails the following steps:

1. A **minimum value of the binder film thickness  $F$**  is adopted as per Sabita Manual 35 which is 5.5  $\mu\text{m}$  (micron), irrespective of mix type or layer. There is no upper limit as an excess of binder will reflect in various other design parameters e.g. voids and VFB.
2. The specific surface area of the aggregate blend is calculated in  $\text{m}^2/\text{kg}$ . Although an empirical concept, it is important for the determination of binder content in the asphalt mix as it relates to the quantity (volume) of binder and the surface area that needs to be coated. The specific surface area ( $SA$ ) of the aggregate particles (in  $\text{m}^2/\text{kg}$ ) is calculated as follows:

$$SA = (2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.30e + 0.60f + 1.6g) \times 0.20482$$

$a$  = 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

3. This calculation is based on a bulk density of the –5 mm fraction of the total aggregate of 2.65  $\text{kg}/\text{m}^3$ . In consideration of the volumetric aspects mentioned above, a correction is required in cases where the bulk density of this aggregate fraction has a different value, as follows:

$$SA_N = SA \times \frac{2.65}{D_{B_5}}$$

Where:

$SA_N$  = Normalised specific surface area

$D_{B_5}$  = Bulk relative density of the fraction passing the 5 mm sieve

4. 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. The sequence of equations (8) to (16) above indicates the determination of the percentage of effective binder  $B_{EF}$ .



The binder film thickness,  $F$ , in  $\mu\text{m}$  (micron) is determined as follows:

$$F = \frac{B_{EF}}{(100 - B)} \cdot \frac{1}{SA_N} \cdot \frac{1000}{D_b}$$

Where:

$B_{EF}$  = Effective binder content expressed as a percentage of the total mass of the mix

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

$BD_B$  = Density of the binder (at 25°C)

$SA_N$  = Normalised surface area



**The total binder content of the mix shall be such that the binder film thickness,  $F$ , based on the effective binder content, is  $\geq 5.5 \mu\text{m}$ .**



In Manual 35, the designer's attention is drawn to the following aspects associated with the determination of the minimum binder content.

1. The specific surface area is essentially 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**.
2. In determining the minimum binder content, the requirements in respect of filler / binder ratio requirements should be given due consideration.
3. 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 of Manual 35.

#### Example

To demonstrate the effectiveness of this approach to account for an adjustment to mass based binder contents for mixes consisting of aggregates from different parent rock, but with similar aggregate shape, texture and identical gradings, consider the following:

Two aggregate blends A & B, consisting of quartzitic sandstone and dolerite with identical gradings and bulk densities of the -5mm fraction, DBA, of 2.65 and 2.8 kg/m<sup>3</sup>, respectively. The calculated surface area, SA for both aggregate blends is 36.2 m<sup>2</sup>/kg.

However, the normalised surface areas differ:

#### **Blend A (quartzitic sandstone)**

$$SA_N = 36.2 \times \frac{2.65}{2.65}$$

$$SA_N = 36.2 \text{ m}^2/\text{kg}$$

#### **Blend B (dolerite)**

$$SA_N = 36.2 \times \frac{2.65}{2.8} \text{ m}^2/\text{kg}$$

$$SA_N = 34.3 \text{ m}^2/\text{kg}$$

Hence, for the same binder film thickness, the proportion by mass of the binder in the mix containing the dolerite aggregate blend would be lower than that of the mix containing the quartzitic sandstone.

#### 4.3.6 PREPARATION OF DESIGN SPECIMENS

For all levels of design the determination of the minimum binder content provides the basis for preparation of specimens for testing, the only difference being that for level IA the Marshall Compaction method may be used – **for all other levels gyratory compaction of test specimens is required.**

Specimens are prepared at three binder contents (as a percent of the mass of the total mix):

1. Minimum binder content based on minimum binder film thickness
2. Minimum binder content +0.5%
3. Minimum binder content + 1.0%
4. Minimum binder content +1.5%



Manual 35 cautions that, at this stage, the filler / binder ratio should be checked for compliance.

Sabita Manual 35



Also it states the number of replicate samples required and the need for loose samples to determine the maximum voidless density.

It also stresses that *mixing* and *compaction* of samples should take place at the correct temperature, based on binder viscosity. The (rotational) viscosity requirements are:

- Mixing: – 0.17 ± 0.02 Pa.s
- Compaction – 0.28 ± 0.03 Pa.s



A number of recommended temperatures for SA mixes and binders are given, although the point is made that the user should ensure that the correct temperature is employed, especially in the cases of mixes with high proportions of RA where comingling of reclaimed and new binder is relied upon and for modified binders, in which case information should be obtained from the manufacturer.

Marshall compaction effort for the Level IA design procedure and the Gyratory compaction effort for all other design levels are covered in Manual35. The standard Marshall method as contained in SANS 3001-AS1 should be followed, except that the recommended number of blows to compact the specimens is 75 blows on the first side + 45 blows on the reverse side.

The number of cycles ( $N_{design}$ ) used in the specimen preparation using gyratory compaction (in accordance with AASHTO T312) depends on the traffic volumes over the project life and, hence, the design level employed as per Tables 24 and 20 of Manual 35, as shown below in Table 9.

**Table 9: Gyratory compaction requirements**

Design Level	Design traffic range (E80)	$N_{design}$
IB	0.3 – 3 million	75
II	3 – 30 million	100
III	> 30 million	125



Manual 35 also states that the designer should strive to attain 4% voids at  $N_{\text{design}}$

To achieve this goal will generally require adjustments to the mix in terms of either:

- aggregate composition
- binder content; or
- both

In making the above adjustments the designer should take care to ensure that:

- permeability of the mix is not adversely affected
- conformance in terms of the minimum binder film thickness is maintained
- filler / binder ratio compliance



During gyratory compaction the workability of the mix is also evaluated as follows: voids of the specimen after 45 gyrations should not exceed the design voids by more than three percent.

Curves of void content vs gyratory cycles for a number of mixes are illustrated in Figure 25. The mix with 5.5% binder, designed for 3 – 30 million E80s, meets the workability requirement of voids at 45 compaction cycles (7%) not exceeding the design voids (4%) at  $N_{\text{design}}$  (100) by more than 3 percentage points.

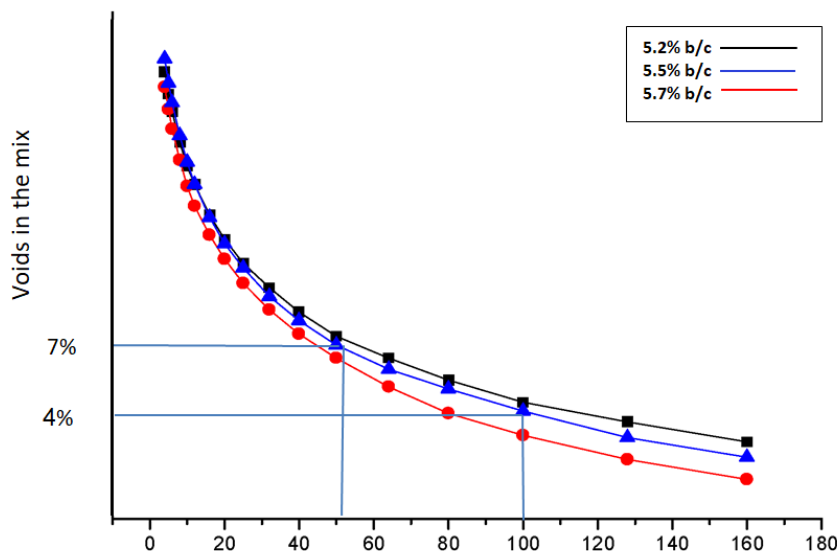


Figure 25: Gyratory compaction curves



Care should therefore be exercised not to pursue resistance to permanent deformation at the expense of fatigue and workability, to obtain a well compacted, durable mix.

*Trial mix evaluation*

#### 4.3.7 TESTING OF DESIGN SPECIMENS

##### Level IA design

Following compaction of specimens in the Marshall apparatus volumetric analysis is carried out as described in Manual 35:

- Two loose asphalt samples for each trial mix are prepared for the determination of the maximum void-less density (MVD) of the mix using Sabita Manual 39 ASP9.
- Determine the bulk density (BD) of the compacted specimens in accordance with ASP8
- 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  $N_{design}$ .

The optimum binder content is determined as follows:

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

Binder content values corresponding to the following requirements are read of from these graphs:

- Voids of 4%
- Peak bulk density
- Peak stability

The optimum binder is determined as the mean value of the individual binder contents determined above as follows, referring to Figure 26:

1. Binder content for peak density: 5.6%
2. Binder content for peak stability: 4.75%
3. Binder content for 4% voids: 5.15%
4. Mean binder content: 5.2%

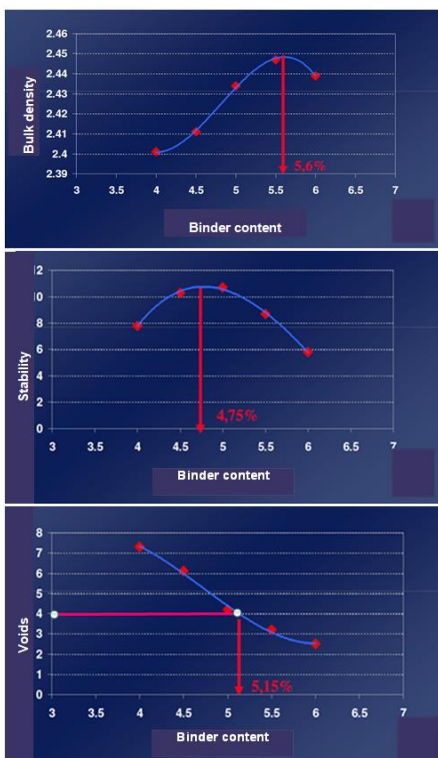


Figure 26: Optimum binder content

*Note on Marshall Stability*





The designer should note that although the determination of the Marshall stability is required in the Level IA design procedure, there are **NO COMPLIANCE REQUIREMENTS** for this property. It has been done away with in Manual 35. The reason for this is that there is little evidence in the literature world wide of a consistent correlation between

Marshall Stability and performance characteristics, particularly resistance of the mix to rutting. A typical research finding is shown in Figure 27 which illustrates this point. In fact some researchers have reported an increase in rutting potential with an increase in stability.

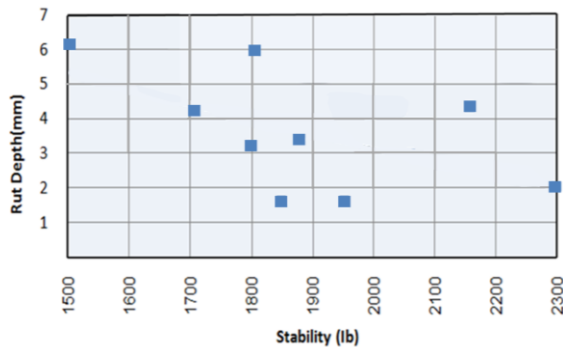


Figure 27: Lack of relationship between Marshall Stability and rutting



The binder content at maximum stability is retained **ONLY AS AN INPUT PARAMETER** to establish the **optimum binder content** of the mix in conjunction with the other considerations covered above. It is reasoned that stability of the mix is a measure of asphalt “strength” deriving from inter-particle friction and cohesion provided by the

bituminous binder. For a given grading / inter-particle friction, a stability peak can reasonably be associated with an optimum / sufficient binder content for mix cohesiveness. Binder content below the peak may be associated with friable mixes; those having binder content in excess of the peak may be over-rich and, hence costly.

### Level IB + design

At these levels of design, the compaction of the specimens takes place using the SuperPave gyratory compactor in accordance with AASHTO T 312. The number of gyrations is given Table 9 above.

Again, two loose asphalt samples for each trial mix are prepared for the determination of the maximum void-less density (MVD) of the mix using Sabita Manual 39 ASP9.

Determine the bulk density (BD) of the compacted specimens in accordance with ASP8.

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  $N_{\text{design}}$ .

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 Tables 22 and 23 of Manual 35 reproduced below.

Select the design aggregate grading and 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  $N_{\text{design}}$ .

### Evaluation of volumetric parameters

Values of the following volumetric entities **at the selected binder content** are then checked against the requirements for voids, VMA and VFB shown in Tables 21 – 23 in Manual 35.

The mix design is acceptable if all the mix design criteria are met. If not, adjustments to aggregate type, grading and / or binder type should be considered.

**Manual 35: Table 21 Voids requirements**

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

**Manual 35: Table 22: Minimum percent VMA**

NMPS (mm)	Minimum VMA* for design voids		
	3%	4%	5%
28	11	12	13
20	12	13	14
14	13	14	15
10	14	15	16

**Manual 35: Table 23: Percent VFB**

Minimum	Maximum
70	80

*Mix acceptance*

Each level of design has difference compliance criteria which should be met in order to make a firm proposal on the mix composition and further assessment with plant trials.

**Level IA**

The only requirements to be met are the volumetric measurements in respect of:

1. Voids in the mix
2. VMA
3. VFB

Where appropriate, the proposed mix can be assessed for durability by measuring the tensile stress ratio (TSR) using the modified Lottman test and assessing the results as per the limits for Level IB.

**Level IB**

At this level of design, both volumetric and performance related criteria should be met. Additionally the durability of the mix in terms of the TSR, using the Modified Lottman test as well as permeability is required to be met; the latter particularly in the case of wearing courses. The performance related criteria here are not fundamental properties; rather surrogate tests are employed that are known to have an empirical relationship with performance characteristics.

Where TSR and permeability criteria are not met, adjustments to the mix design should be made. These may typically entail the use of hydrated lime, anti-stripping fluids or changing the aggregate source in the case of insufficient durability. Permeability may require a re-examination of the mix composition in terms of mix type.

Table 10 below – a reproduction of Manual 35, Table 28 – shows a summary of the design criteria at this level.

**Table 10: Design criteria for Level IB**

Property	Test	Method	Criteria
Durability/TSR	Modified Lottman	ASTM D 4867 M	See Table 27 (Manual 35)
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	TRH8 (CSRA 1987) See Appendix B	$\leq 1 \times 10^{-8} \text{ cm}^2$

**Levels II and III**



The design procedures at these levels involve fairly advanced procedures and should be attempted only by experienced practitioners. Here fairly direct measurements are made to assess the damage and response parameters of mixes.

*Dynamic modulus*

Dynamic modulus test is conducted at frequency sweeps of 0.1, 0.5, 1, 5, 10, and 25 Hz.

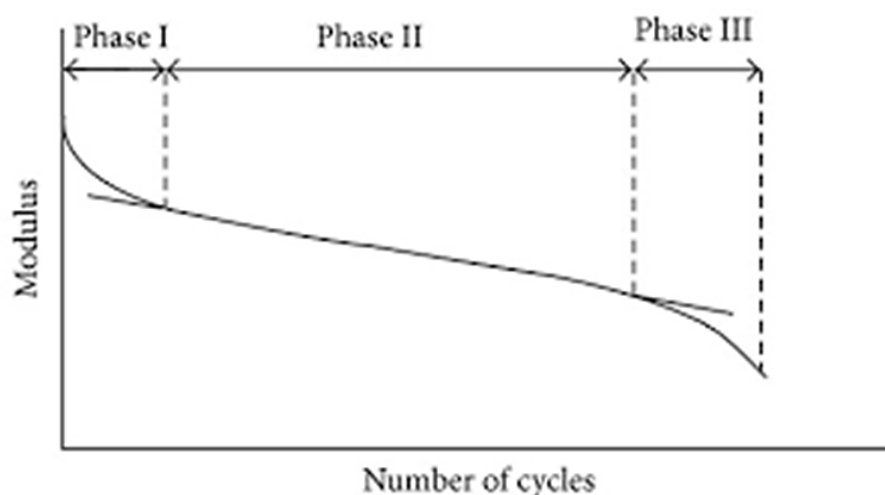
For Level II, the sweep is carried out at one test temperature of 20°C.

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).

*Fatigue*



The effective stiffness of asphalt mixtures decreases throughout the crack developing process in pavements. Generally, a plot of the stiffness versus loading cycle of an asphalt mixture during fatigue testing exhibits three regimes of evolution, as shown in Figure 28.



**Figure 28: Evolution of stiffness under fatigue testing**

In Phase I, a rapid decrease in stiffness can be observed, followed by Phase II, which corresponds to a linear decrease in stiffness with respect to the number of load cycles. In Phase III, fracture cracking occurs as a result of the damage acceleration of micro-cracks. Ultimately these develop into observable macro-cracks, which cause the failure of the specimen.

For Level II the fatigue test is carried out at one test temperature of 10°C and a loading frequency of 10 Hz at three strain levels to generate fatigue curve for the mix.

For Level III the fatigue test is carried out at three test temperatures of 5, 10 and 20°C at 10 Hz at three strain levels to generate isotherm fatigue curves for the mix.

Fatigue failure is defined as the load cycle at which the product of the specimen stiffness and loading cycles ( $S \times n$ ) is a maximum as illustrated in Figure 29.

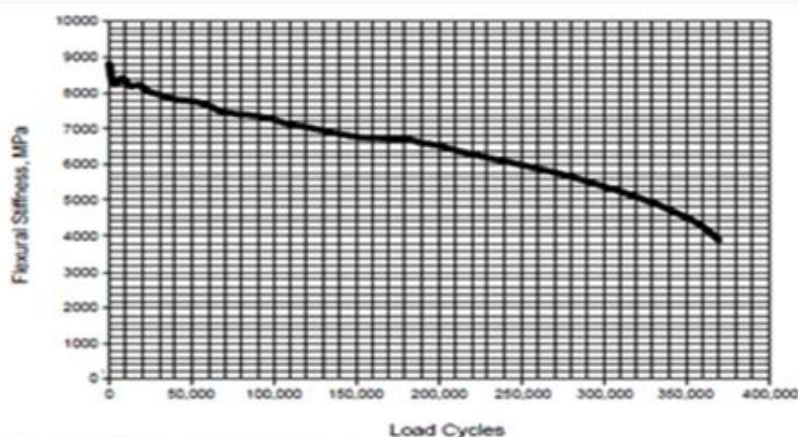


Figure 6—Stiffness versus Load Cycles

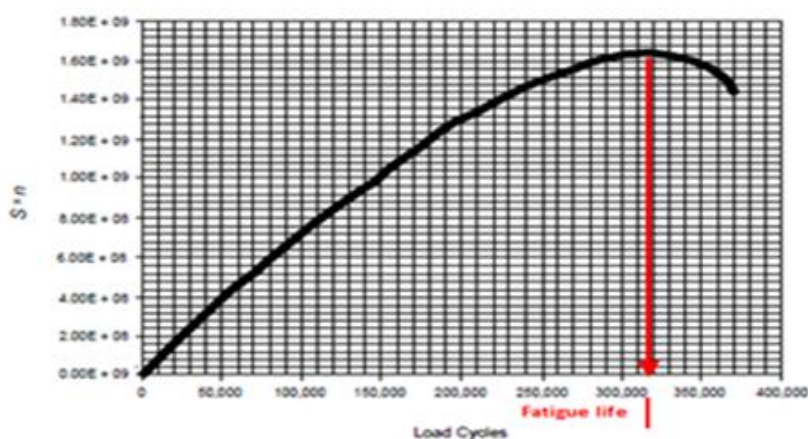


Figure 29: Determination of fatigue life

### Rutting resistance

This requirement using the Hamburg Wheel Tracking Test is clearly set out in Manual 35 and needs no further comment other than stating that the compliance criteria are based on both design traffic and the relevant climatic region. These are set out in Table 30 of Manual 35.

Importantly, it is noted in Manual 35 that **recording and assessing mix performance on the basis of the final data after the appropriate number of wheel passes only might be misleading.**



Factors such as the post compaction consolidation, the creep slope (number of wheel passes per mm rut depth) as well as the stripping slope should be considered when assessing data. (See Figure 30)

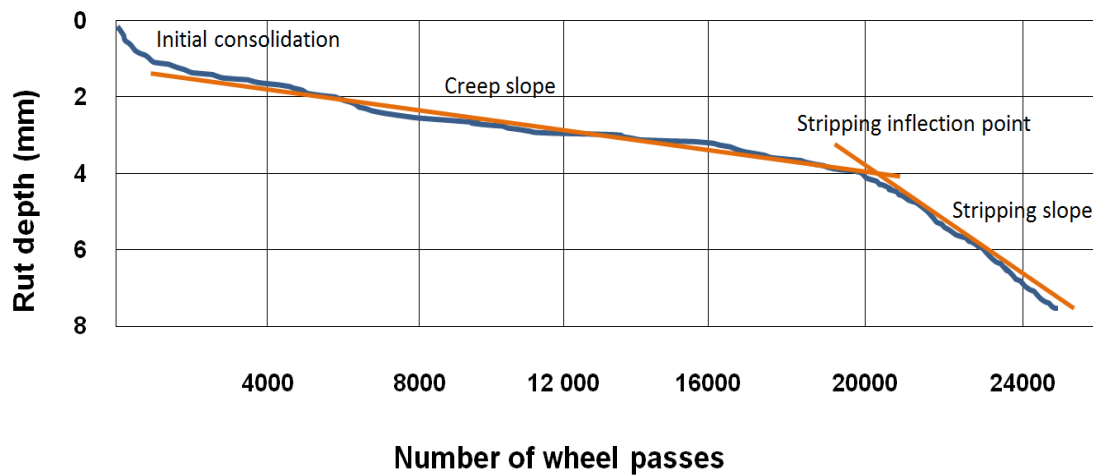


Figure 30: Definition of HWTT phases

A summary of the tests carried out at Design Levels II and III, together with the number of specimens, is shown in Table 11.

Table 11: Performance tests for Levels II & III.

Property	Test conditions	No. of specimens	Test method
Workability	Superpave gyratory compactor, air voids after specified number of gyrations	3	AASHTO T312
Durability	Modified Lottman test conditions	6	ASTM D 4867M
Stiffness/ (dynamic modulus)	AMPT dynamic modulus at temperatures of -5, 5, 20, 40, 55°C; loading frequencies of 25, 10, 5, 1, 0.5, 0.1 Hz	5	AASHTO TP 79
Permanent deformation	HWTT at relevant number of passes.	2 <sup>1</sup>	AASHTO T 324
Fatigue	Four-point beam fatigue test at maximum of three strain levels and three temperatures.	9 <sup>2</sup>	AASHTO T 321

<sup>1</sup> Number of specimens per binder content

<sup>2</sup> This is the maximum number of specimens required for Level II design. For Level III design it may be necessary to prepare 18 specimens to enable the designer to repeat a test at a specific temperature and / or strain level.

**NOTE OF CAUTION**



Regarding the adoption of appropriate limits for, particularly dynamic modulus, fatigue life and resistance to rutting, an approach which links these key response and damage characteristics to the pavement structural requirements is recommended. The design of the asphalt at Levels II & III cannot proceed without due consideration of the requirements pertaining to the design situation. In other words, the requirements for flexural stiffness, fatigue life and rut resistance should be fully integrated with either the project at hand or for a range of operating conditions of:

- climate
- traffic
- pavement (substrate) response.

This principle is illustrated in Figure 31 which presents a hypothetical example of the mix selection process for medium to high volume roads. The performance requirements for the mix are determined based on mechanistic-empirical pavement analysis. In this example, requirements are set for:

- Stiffness,
- Fatigue, and
- Permanent deformation (rutting).



These requirements can then be included in the tender documentation, together with e.g. requirements for workability (i.e. air voids at a standard number of gyrations in gyratory compactor) and durability (tensile strength ratio in the modified Lottman test). Functional requirements such as skid resistance for the mix would also be specified for wearing course asphalt.

The figure illustrates that “Mix 2” meets the structural requirements of the layer optimally.

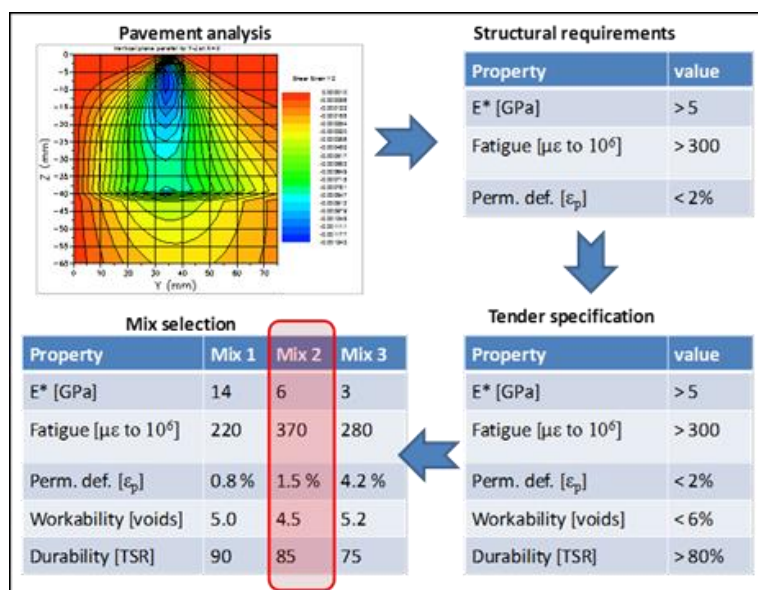


Figure 31: Hypothetical example of mix selection to meet pavement performance criteria

## Fatigue life



There is a complex interaction between temperature, on the one hand, and asphalt pavement material response and the onset of fatigue damage, on the other. For example, an increase in pavement temperature may decrease layer stiffness, giving rise to increased (tensile) strains and, hence layer susceptibility to fatigue distress.

However, this postulation has not been observed and it would be difficult to confirm through the adoption of a single figure for fatigue life. The reason is simply that fatigue life also increases with temperature as is illustrated for a particular mix in Figure 32. In this case the fatigue life of the layer at 30°C is approximately 6 times the life at 10°C – a significant difference.

It is therefore recommended that, in cases warranting the adoption of Design Level III, the following type of approach is adopted:

1. Develop master curves for asphalt stiffness (dynamic modulus) and determine characteristic curves for an appropriate set of temperatures.
2. Interpose these with the relevant fatigue isotherms to assess the fatigue life with respect to the requirements of the structural design parameters.



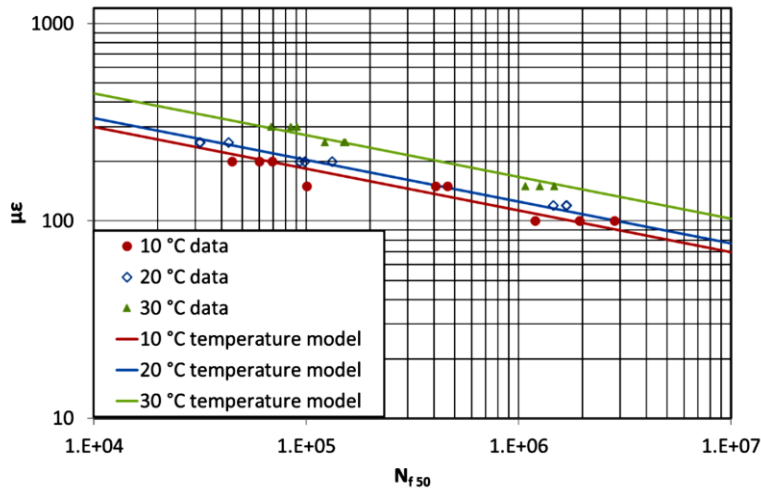


Figure 32: Fatigue life at various temperatures

Source: AAPA

## 4.4 DESIGN OF SPECIAL MIXES

Sabita Manual 35 presents a design procedure for asphalt in general and mentions that special considerations for specific mix types are presented in other publications. These are listed below with key considerations that should be taken into account when designing such mixes.

### 4.4.1 COLD MIXES

Two categories of product fall within this ambit:

- cold asphalt proprietary products; and
- generic technologies associated with bitumen stabilised materials using either bitumen emulsion or foamed bitumen processes.

As regards the former, i.e. cold asphalt, it is recommended that only Agrèment South Africa accredited products be considered. This process of accreditation of various commercially available products for a range of applications is currently underway.

The reference document for bitumen stabilised materials – TG2: Bitumen Stabilised Materials has been recently revised and updated.

Note:



While it may be argued that bitumen stabilised materials are not asphalt types as such, particularly as a result of the typically lower binder contents compared to asphalt and thus different behaviour, they are included here for the sake of completeness.

TG2 Bituminous  
Stabilised Materials



### 4.4.2 POROUS ASPHALT

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

Sabita Manual 17



Porous asphalt are stone skeleton mixes usually of nominal stone sizes of 10 mm or 14 mm with low fine aggregate contents (approximately 11% passing the 2 mm sieve) and containing between 3,0 and 4,0 % of filler to obtain the required void content in the region of 20%.

Binder film thickness is usually high for improved durability. To avoid drain-down of the binder, especially during transport and paving, it either contains fibre additives or is modified to raise the effective viscosity of the binder. In some cases both measures are adopted.



A special feature of the design process is the determination of the optimum binder content. The design binder content is selected as the average of the higher of the minimum binder contents (durability and abrasion resistance according to the Cantabro test) and the lower of the maximum binder contents (void content and binder run-off).

The results of the procedures involved in the determination of the optimum binder content are illustrated diagrammatically in Figure 33.

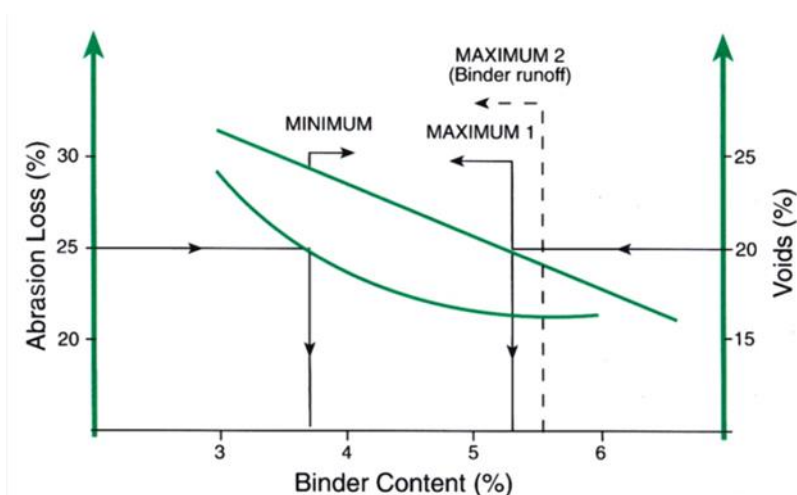


Figure 33: Optimum binder content for porous asphalt

#### 4.4.3 MIXES FOR LIGHT TRAFFIC IN RESIDENTIAL AREAS

The reference document for this application of asphalt is: Sabita Manual 27: *Guideline for thin layer hot mix asphalt wearing courses of residential streets.*

Sabita Manual 27



For clarity, thin layer hotmix asphalt is defined as layers less than 30 mm thick that:

- Carry moderate to light traffic at low speeds that function as a surface treatment offering protection against traction and braking forces imposed by vehicular traffic, rather than contributing measurably to the structural capacity of the pavement;
- Have sufficient resilience to provide a durable surface in the face of prevailing transient deflections;
- Protect the underlying pavement layers against the ingress of water, thereby protecting the integrity of layer materials; and
- Provide an appropriate degree of skid resistance through finished texture.



In most cases, asphalt manufacturers have standard mixes for these applications and the manual recommends that they be approached to supply asphalt in terms of the specific requirements.



Thin layers will be very susceptible to rapid cooling, which will militate against the achievement of adequate compaction. **Consequently, extra care should be taken in both the design and construction procedures to ensure that adequate densification will be**



**achieved.** Also, the finish of the base course may not be of the highest quality, hence layer thickness control may prove to be a challenge.

Manual 27 sets out the following unique mix design criteria, most of which are unique to this type of asphalt:

- Low permeability, through limited and dispersed voids, to protect underlying layers – often granular bases – from the ingress of water;
- Compactability, given the rapid cooling of thin layers and, hence, the limited compaction windows. Two compositional aspects that would require attention are appropriate maximum aggregate sizes and binder grades;
- A surface texture to provide sufficient skid resistance associated with low speeds (<80 km/h). In view of the generally low prevailing speeds to be accommodated, the skid resistance would be derived from the micro-texture of the asphalt;
- A compliant consistency, being sufficiently flexible and durable to accommodate the transient deflections associated with light, mainly granular, pavement structures rather than meeting structural requirements e.g. stiffness (load-spreading capacity) and resistance to permanent deformation.



To meet these criteria, it is recommended that *sand-skeleton* type mixes are used for thin layer asphalt in light traffic urban environments. By this, it is meant that the load is carried primarily by intergranular friction of the <2 mm fraction of the mix. In such cases the volume of mastic is limited to ensure that the integrity of the sand skeleton structure is not adversely affected.

Seen in conjunction with the binder grade, Table 11 indicates how sand skeleton mix types and a soft binder will facilitate the achievement of the design objectives.

**Table 12: Achievement of design objectives**

Design objective	Sand skeleton	Softer grade of bitumen
Low permeability	✓	
Compactability	✓	✓
Low speed skid resistance	✓	
Flexibility	✓	✓

The manual makes the important point that the determination of densities of these thin layers is impractical; hence compaction control should focus on proven method statements of roller compaction, as well as measurement of permeability.



**The use of a soft binder grade is strongly recommended as it prolongs the compaction window**, especially under adverse conditions of low temperatures and wind. As the layer is not expected to carry heavy loads and, due to its limited thickness, it is most unlikely to undergo permanent deformation, which is sometimes associated with soft binder grades.

#### 4.4.4 WARM MIX ASPHALT

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

Sabita Manual 32





Warm mix asphalt (WMA) involves the manufacturing and paving of asphalt mixes at significantly lower temperatures than conventional hot-mix asphalt (HMA), while at least matching the quality of conventional HMA mixes. As a consequence there are a number of health, environmental and engineering benefits associated with the use of WMA. These are listed in Manual 32.

WMA can be conveniently classified by the degree of temperature reduction compared to that of conventional HMA. This is illustrated in Figure 34, which shows the typical ranges in mix temperature, from cold mixes to conventional HMA. It also shows how the consumption of fuel increases to produce mixes at higher temperatures.

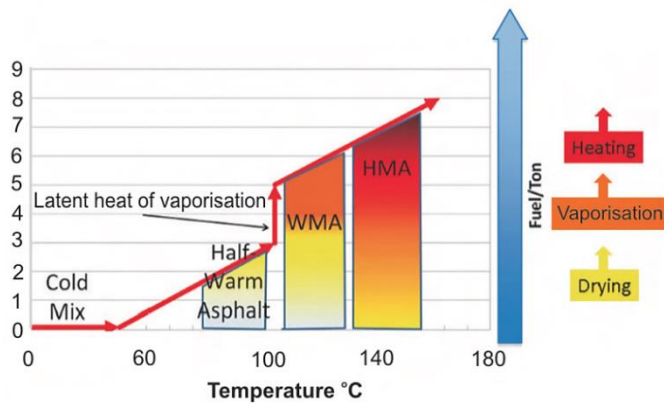


Figure 34: Temperature ranges of asphalt manufacture

While HMA is generally manufactured at temperatures between 140°C and 160°C, WMA is typically produced at temperatures between 100°C and 140°C.

A number of technologies are employed in the manufacture of WMA, essentially to enable proper coating of aggregates and mix workability at lower temperatures. The products are either introduced as ready to use binders or additives during the mixing process. The warm mix technologies fall within three categories:

1. Water technologies
2. Chemical additives
3. Rheological modifiers

Subcategories of the above are described in some detail in Manual 32.



**Manual 35 alerts the designer to note that it is important that the final binder properties are not adversely influenced by the WMA additives. It suggests that binders be evaluated with the additives already present.**

Sabita Manual 35



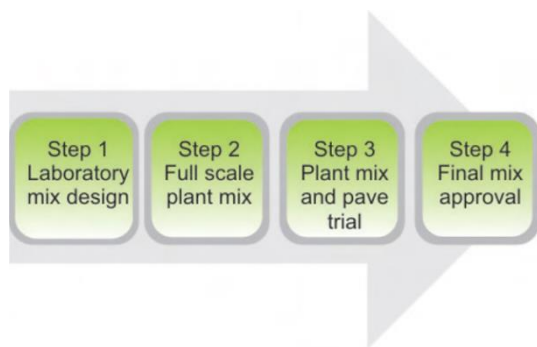


Figure 35: Mix design process

The complete process – from laboratory design to final mix approval – is depicted in Figure 35.



During the **laboratory design process** it should be noted that, due to the lower temperatures of production, there is less chance for the aggregates and RA to be fully dried out and particular attention should be given to the mix's moisture susceptibility. Consideration should be given to the inclusion of 1% hydrated lime or amine adhesion promoter in the mix and, in addition, it may be necessary to add an anti-stripping agent, depending on the bitumen adhesion properties of the aggregate.

The next step is the **full scale asphalt plant mix**. Apart from the usual step of e.g. assessing the “shift” between laboratory and plant specimens, it is important that the plant capabilities be assessed. Manual 32 lists six important facets that need to be addressed.

During the **plant mix and paving trial** sufficient quantities of plant mixed WMA are used to pave in a trial section. This final step in the mix approval process is intended to provide confidence that production efficiency is achieved, adjustments enable target reductions in temperature, ancillary equipment and downstream operations are calibrated and functioning properly.

#### 4.4.5 EME

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

Sabita Manual 33



The design procedure presented in Manual 33 is based on the French procedure for *Enrobé à Module Élevé*. In transferring the technology to South Africa, due care was taken not to alter key elements of the design procedures and criteria, other than using available laboratory equipment to determine performance characteristics, following replicating studies.

Key performance qualities of EME are:

1. Very high stiffness to reduce tensile strains in the layer and effect superior load transfer to substrates, thereby significantly reducing the stresses experienced by underlying layers
2. Relatively high binder content which, in association with relatively low tensile strains, will enhance fatigue life.
3. The high stiffness binder used in EME (10/15 pen grade), in conjunction with a well-developed aggregate structure offers high resistance to rutting.

These three features address the conflicting design objectives (of rut resistance and fatigue / durability), mentioned at the beginning of this section, quite effectively.



As the use of EME, being costly in view of the high proportion of a special bituminous binder, is confined to routes of high importance with design traffic in excess of 50 million ESALs, its design should best be carried out by experienced practitioners.

#### 4.4.6 MIXES WITH RECLAIMED ASPHALT

Reference document: Sabita Manual 36 / TRH 21: 2016: *Use of reclaimed asphalt in the production of asphalt.*

Sabita Manual 36 /  
TRH21



As the use of proportions of reclaimed asphalt (RA) in asphalt has steadily increased over the last decade, the design of mixes with more than 10% nominal proportions of RA is now common practice and should form part of the skills profile of asphalt designers.

Manual 36 describes a fairly extensive design process covering a number of steps e.g. investigating reclaimed asphalt (RA) sources, field investigations, reclaiming and processing procedures, design procedure, mixing plant requirements, trials and quality control as well as aspects associated with contractual matters, sustainable practice and health and safety considerations.



Briefly, the aim of the mix design procedure is to determine the proportions of new aggregate, new binder and RA that will fulfil the requirements of the specification. The point is made that the RA content to be introduced should be determined by mix design characteristics, not by a particular asphalt plant's capability or perceptions on the part of the client maximise the use of milled material available on a specific contract.

A number of design elements are unique to mixes with RA and are briefly introduced here.

#### RA and binder proportions



The RA content is determined by considerations of the amount of binder in the RA, expressed as a proportion of the total binder in the mix, and termed "RA binder replacement".

The RA binder replacement is determined as follows:

$$\text{Total RA binder in the mix, } B_{RA} = \sum_{i=1}^n \left( \frac{(\text{RA content})_i}{100} \times \frac{(\text{RA binder content})_i}{100} \times 100 \right)_{f_i}$$

Where:

$n$  = total number of processed RA fractions used in the RA mix design

$f_i$  = RA fraction number, e.g.  $f_1, f_2$  etc.

$B_T$  = Target (optimum) binder content of the mix  
i.e. the sum of RA binder and virgin binder

$$\text{RA binder replacement, } B_R = \frac{B_{RA}}{B_T} \times 100$$

$$\text{Virgin binder required in mix, } B_V = B_T - B_{RA}$$

Where:

$B_T$  = Target (optimum) binder content of the mix  
i.e. the sum of RA binder and virgin binder



The proportion of RA aggregate to the total aggregate in the mix is closely related to the proportion of RA in the mix. Consequently the **aggregate testing guidelines** are based on the RA content as shown in Table 13.

**Table 13: Guideline for tests on quality of aggregate in RA**

RA content in mix	RA Aggregate quality
≤ 15%	<ul style="list-style-type: none"> <li>• Check intrinsic aggregate properties;</li> <li>• Aggregate grading</li> </ul>
> 15%	<ul style="list-style-type: none"> <li>• Check intrinsic aggregate properties;</li> <li>• Aggregate grading;</li> <li>• Coarse aggregate strength (ACV, 10% FACT);</li> <li>• Flakiness index</li> </ul>



The proportion of RA binder, as described by the *RA Binder replacement factor*, determines **the tests required for the recovered RA binder**, as shown in Table 14.

**Table 14: Guidelines for tests on quality of the recovered binder**

RA Binder replacement (%)	Recovered Binder Tests
≤ 15	<ul style="list-style-type: none"> <li>• Binder content;</li> <li>• Contamination (e.g. coal tar)</li> </ul>
> 15	<ul style="list-style-type: none"> <li>• Binder content;</li> <li>• Softening Point;</li> <li>• Penetration;</li> <li>• Contaminants (e.g. coal tar);</li> <li>• Where required (in terms of PG specification)               <ul style="list-style-type: none"> <li>- G*, δ, J<sub>NR</sub>, S and m</li> </ul> </li> </ul>

### Preparation of the RA sample

The preparation of the RA before commencing with the mix design depends on the proportion of RA that is intended to be added to the mix. Manual 36 provides guidelines (in terms of the percentage RA binder replacement) for the preparation and testing of RA before carrying out the preliminary mix design.



It is noted that the fractionating of the RA will enhance its uniformity and is strongly recommended. Note that the actual screen sizes for separating RA fractions should be left to the asphalt producer to optimise the use and quality of the RA.

In the case of mixes with **RA contents higher than 15%**, aggregate blends should be carried out to establish the most effective one to produce the required aggregate packing (grading) of the intended mix.

In the case of mixes where the **RA binder replacement is 15% or higher**, once the gradation has been determined, the binder should be extracted to determine the bitumen properties of the RA binder and, ultimately, the binder blend.



The binder should be designed to meet the binder requirements of final mix. To meet these requirements, it may be necessary to use a soft binder or rejuvenator. Usually softer grades do not have enough co-mingling potential when the percentage RA binder

replacement exceeds 15% and the use of rejuvenator agents should be considered. When the RA binder replacement is higher than 25%, rejuvenator agents would generally be required.

Once again, it should be stressed that only a few features of the complete design process has been presented here; the designer should consult Sabita Manual 36 for a complete description of the design process.

#### 4.5 CONSTRUCTION AND QUALITY ASSURANCE

As stated in Sabita Manual 35, it is recommended practice that, after the successful design of a new mix in a laboratory, a trial mix is produced to assess workability and comparison of *in situ* properties of the mix with those of the laboratory produced specimens. Upon successful completion of the trial section, plant production and, where necessary, adjustment to the mix proportions, paving commences as per contractual requirements.

Process and acceptance control procedures to ensure consistent quality are dealt with comprehensively in Manual 35 as well as Sabita Manual 5: *Guidelines for the manufacture and construction of hot mix asphalt*.



What is worth mentioning here is that the key goal of a quality assurance program is to ensure that the component materials – bituminous binders, aggregates and filler – in the laid mat comply with the stated requirements in supply agreements, and, hence, do not differ significantly in quality from those used in project designs, in terms of both their type and proportions as finalised after the plant trials. This should be the focus of any quality assurance process, since, if this goal is achieved, it follows that the expected performance characteristics would materialise.

Operational procedures necessary to control quality arising during transport, off-loading and storage of bitumen and bituminous products are covered in Sabita Manual 25: *Code of Practice: Transportation, off-loading and storage of bitumen and bituminous products*.

Sabita Manual 25



Regular, day-to-day plant control and tests that should be carried out and communicated to the designer cover:

- Stockpile and cold-feed gradations;
- Cold-feed proportioning adjustments;
- Binder quality and content;
- Gradation of aggregate in mix;
- Adjustments to mixing time and temperature;
- Preparation of laboratory specimens for the relevant testing of compositional and volumetric parameters.

## APPENDIX A MAXIMUM DENSITY GRADATIONS

In 1907 Fuller developed a form of presenting particle size distribution (grading) in a manner which was thought to be a convenient form for determining the grading line which would represent maximum density, which can then be used as a basis for adjustments to the grading for specific purposes in both asphalt and concrete design. The expression used was:

$$p_i = \left(\frac{d_i}{D}\right)^n$$

Where:

$p_i$  = percent passing  $i^{\text{th}}$  sieve

$d_i$  = opening size of the  $i^{\text{th}}$  sieve

$D$  = maximum particle size

Further work suggested that, whereas the original Fuller expression adopted an exponent of 0.5 which was relevant to spherical particles, a value of  $n = 0.45$  would simulate the packing of (angular) aggregate particles more effectively.

Consequently in the 1960s the FHWA introduced the standard gradation graph widely used in the asphalt industry today where the exponent of  $n = 0.45$  was adopted, i.e.:

$$p_i = \left(\frac{d_i}{D}\right)^{0.45}$$

This graph is slightly different than other gradation graphs because it uses the **sieve size raised to the power 0.45 as the x-axis units**. Thus these proportions passing the individual sieve sizes fall on a straight diagonal line from zero to 100 for the maximum aggregate size of the mixture being considered as shown in Figure 36. The exact location of this line is somewhat debatable, but the locations shown in Figure 36 are generally accepted.

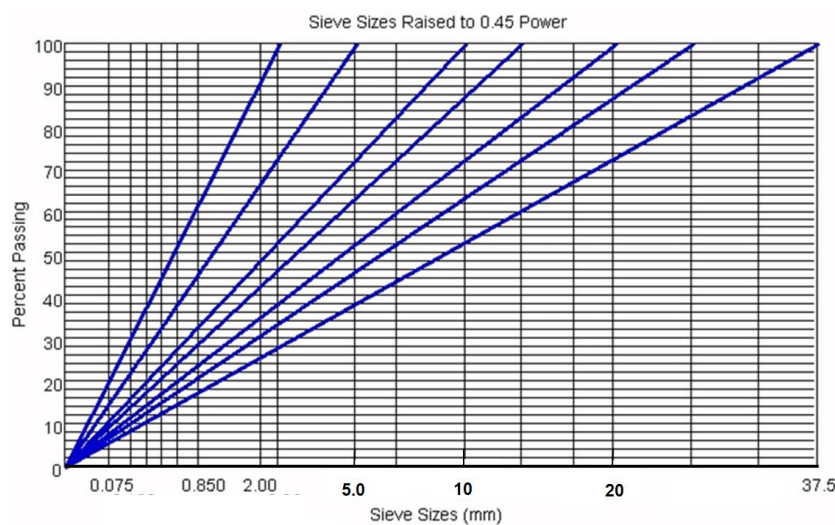


Figure 36: Maximum density lines for  $n = 0.45$

The point was made then that adopting aggregate gradings (with maximum packing / nesting density) close to these maximum density lines may provide inadequate space for adequate binder concentration while providing sufficient voids. Consequently optimal packing adopted for asphalt mixes results in gradings that will deviate from these lines, as illustrated in section 3.1 of the main text.



## APPENDIX B PERMEABILITY

### TRH 8 (1987) APPENDIX C: Determination of the air permeability of compacted Marshall specimens<sup>9</sup>

#### Scope

The measurement of air permeability is non-destructive and may be performed on laboratory-compacted Marshall specimens or cores removed from a pavement.

#### Apparatus

Model AP-400 A, Asphalt Paving Meter, is a self-contained device capable of measuring air flow rates of up to 5 l per minute at low pressure differentials.

The device has a maximum volume capacity of 1 litre. It is equipped with two calibrated sight tubes; one measures in 25 ml increments for dense specimens, and the other is graduated in 500 ml increments for open, permeable specimens.

Incorporated in the unit are the valves required to perform the test: a pressure squeeze bulb for filling the test system with water; a sensitive manometer for pressure setting; a metal cylindrical jacket slightly larger than the 101.6 mm diameter Marshall specimen and containing a 101.6 mm diameter rubber triaxial sleeve as an inner liner; a domed test cap; and a specimen support.

#### Method

1. Remove either stem valve by screwing counter-clockwise and lifting out of the case. Set the main control valve at 'exhaust' or 'test'. Fill the unit with 2 000 ml of water through the recess left by the stem valve.
2. Replace the stem valve in its mounting.
3. Fill one of the two calibrated volumetric cylinders in the following manner:

#### NOTE:

1 000 ml cylinder is used for open specimens.

300 ml cylinder is used for dense specimens.

- (a) Push the main control valve to the 'off' position.
- (b) Open the stem valve above the selected cylinder by turning counter-clockwise. One revolution is adequate. Check to make certain that the volumetric cylinder stem valve not being used is closed tightly. The air flow valve on top of the rubber bulb assembly should be open (unscrewed about two turns).
- (c) Squeeze the rubber bulb, pumping air into the reservoir. The pressure will force water from the reservoir into the measuring cylinder, as can be seen by the rising water-level in the sight tube. Continue pumping until the sight tube is filled to within approximately 25 mm of the top. Close the stem valve connected to this tube.
- (d) Turn the main valve to the 'exhaust' position to allow excess air pressure to escape from the reservoir.

Connect the rubber tube of the specimen test cap to the pressure port located in the lower left corner of the instrument. Connect the second tube of the specimen test cap to the manometer

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<sup>9</sup> This method is currently being revised and updated and will be published as ASP5 in Sabita Manual 39.

port located at the top left of the manometer. (The rubber tubes of the test cap are interchangeable and may be connected to either the pressure port or the manometer port.)

4. Level the instrument by observing the level bubble and adjust the instrument on the swivel tripod.
5. Set the manometer at the 'zero' reading by loosening the screw at the bottom of the manometer and sliding the scale until the 'zero' reading coincides with the level of the manometer fluid in the nearly horizontal tube of the manometer.  
Be sure that the manometer valves are in an open position while zeroing the manometer and during a test.
6. The Marshall specimen is placed on the support and the metal cylinder, with the rubber triaxial sleeve in place, is slipped over both. The domed test cap with rubber tubes attached is then placed over the top of the specimen and the rubber -sleeve inflated with the hand bulb to seal the cap, specimen and support. If the vertical face of the specimen is rough, it may be sealed with paraffin wax prior to placing in cylinder.
7. The instrument is now ready to perform the test: the main valve is set at 'exhaust', the measuring cylinder is full, both manometer valves are open, the manometer is reading 'zero' and the test cap is sealed to the specimen.
8. Proceed as follows:
  - (a) Push the main control valve to the 'test' position. Close the airflow valve at the top of rubber assembly.
  - (b) Open the stem valve of the filled volumetric cylinder slowly, allowing water to flow from the cylinder back to the reservoir. Adjust the stem valve so that a constant pressure is developed and recorded by the manometer. Normally, tests are run at 6.4 mm of water pressure. The pressure is held constant throughout the test by minor adjustments to the stem valve.
  - (c) Once constant pressure has been attained, the time required for water to flow between two of the calibration marks on the sight tube is measured accurately.  
A volume of flow sufficiently large to require a time interval of at least 20 seconds should be selected to obtain accurately repeatable results. Time intervals exceeding 2 minutes do not add to the accuracy of the test and only prolong the testing time.
9. A minimum of three Marshall specimens should be tested at each binder content and the mean rate of air flow through the specimens taken

### Calculation

The air permeability is then calculated using the following formula:

$$K = 2.0331 \times 10^{-9} \frac{QL}{h}$$

Where:

K = fundamental permeability (cm<sup>2</sup>)

Q = rate of air flow through specimen (cm<sup>3</sup>/s)

L = average height of specimen (cm)

h = air pressure difference expressed in height of water column (cm)

It should be noted that the above formula applies only to a 101.6 mm diameter cylindrical specimen.

## APPENDIX C GRADATION TERMINOLOGY

Several terms are commonly used to describe particle size distribution or gradation. These are not precise technical terms. Rather they describe families of aggregate blend compositions that are distinct in particle size distribution.

### **Continuously graded**

A type (also termed *Dense* or *Well-Graded*) that refers to a gradation that is near the  $n = 0.45$  power curve (See Appendix A Maximum density gradations) for maximum density. Many mixes used in SA conform to this type of gradation and typically are near the 0.45 power curve but not right on it. Generally, a true maximum density gradation (exactly on the 0.45 power curve) would result in unacceptably low VMA.

### **Gap Graded**

This term refers to a gradation that contains only a small proportion of aggregate particles in the mid-size range. Hence the grading curve is flat in the mid-size range. This type of grading is considered where permeability of the mix is a critical requirement as voids in a fine aggregate matrix are well dispersed. Mixes of this type typically require some texturing, e.g. by spreading pre-coated chippings on the paved mat prior to rolling.

### **Open Graded**

Open graded aggregate blends contain only a small proportion of fine aggregates. This results in more air voids because there are not enough small particles to fill in the voids between the larger particles. The curve is near vertical in the mid-size range and flat and near-zero in the small-size range. Open graded blends are used for porous asphalt and thin friction courses.

### **Uniformly Graded**

This type of grading defines single sized aggregate i.e. particles in a very narrow size range. In essence, all the particles are the same size. The curve is steep and only occupies the narrow size range specified. It would generally apply to aggregate fractions for blending in asphalt mixes.